CALIBRATION OF THE HCM 2010 ROUNDABOUT CAPACITY EQUATIONS FOR GEORGIA CONDITIONS

A Thesis
Presented to
The Academic Faculty

by

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In Partial Fulfillment
of the Requirements for the Degree
Masters of Science in the
School of Civil and Environmental Engineering

Georgia Institute of Technology
August 2012

CALIBRATION OF THE HCM 2010 ROUNDABOUT CAPACITY EQUATIONS FOR GEORGIA CONDITIONS

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To Mom and Dad.

ACKNOWLEDGEMENTS

There are many people that I wish to acknowledge and thank. First, I would like to thank my thesis reading committee Dr. Michael Hunter, Dr. Michael Rodgers, and Dr. Kari Watkins. I would especially like to thank my advisor, Dr. Michael Hunter, for his support throughout this process and for giving me the opportunity to work on this research.

I would like to thank everyone in the lab that helped me with the different pieces of this research. Lance Ballard and Chenhao Liu helped me collect data. Lakshmi Pessapati developed the program that we used for collecting vehicle timestamps. Dwayne Hencelwood helped me write scripts in R to analyze my data. Aaron Greenwood drew the diagrams for follow-up headway and critical headway. I would especially like to thank Wonho Suh for his help with data collection, data analysis, and everything else. Lastly, I would like to thank my parents and my brother Ben for all of their continual love and support.

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SUMMARY

There is increased interest in building modern roundabouts in Georgia and in the United States because of the safety and operational benefits that can be realized from this type of intersection. With this interest comes the increasing need to be able to estimate the capacity that a roundabout can provide after it is built. In the early 2000s, a National Cooperative Highway Research Program (NCHRP) study was conducted that, among other tasks, developed single-lane and multi-lane roundabout capacity estimation equations. These equations, presented in the Highway Capacity Manual 2010 (HCM 2010), can be calibrated using locally determined values of follow-up headway and critical headway.

This study was designed to calibrate the HCM 2010 roundabout capacity equation for single-lane roundabouts to driving conditions in Georgia. In order to develop estimates of the calibration parameters, video imagery was recorded for 13 approaches at six roundabouts in Georgia for approximately two hours during the peak period. A total of 29.5 hours of video was recorded. Data from three of these roundabouts forms the basis of this thesis. The videos were processed by a Java program to collect time stamps that were subsequently used in Microsoft Excel® to calculate the follow-up and critical headway values required for calibration.

The values of critical headway and follow-up headway that were found from the video data are presented in the results as well as the single-lane capacity equations calibrated from the data. Two types of analysis were done, one that includes exiting vehicles and one that does not include exiting vehicles. When exiting vehicles were

excluded, the weighted average of follow-up and critical headway were found to be 3.46 and 4.17 seconds respectively and when exiting vehicles were included in the analysis the weighted averages of the follow-up and critical headway were found to be 2.80 seconds and 3.34 seconds respectively. It was found that exiting vehicles do have an impact on the operations at the roundabout in most cases, and including exiting vehicles in the analysis tends to increase the capacity predicted by the calibrated equations.

CHAPTER 1

INTRODUCTION

Roundabouts are relatively new in the United States compared to other countries. However, there is a rising interest in roundabouts due to the operational and safety benefits provided by this type of intersection. When weighting alternative intersection designs, capacity is a major factor and thus the ability to accurately predict the capacity of a roundabout is important. The current method for determining the capacity of roundabouts in the United States is found in the Highway Capacity Manual 2010 [2]. This method was developed as part of a National Cooperative Highway Research Program (NCHRP) study [3]. The capacity equations that resulted from the NCHRP study use critical headway and follow-up headway to determine capacity on an approach basis. The default capacity equations can be used as they appear in the HCM 2010 or they can be calibrated to local conditions. This effort seeks to calibrate the default HCM 2010 capacity equations to Georgia conditions.

1.1 Background

HCM 2010 defines roundabouts as, "intersections with a generally circular shape characterized by yield on entry and circulation around a central island (counterclockwise in the United States)" [3]. Multi-lane roundabouts are defined as, "roundabouts [that] have more than one lane on at least one entry and at least part of the circulatory roadway" [3]. There are other types of circular intersections that do not fit these descriptions, such as traffic circles and rotaries. The modern roundabout was created decades after the first circular intersections were built and it evolved from the challenges that affected the operations of its predecessors [2].

1.1.1 The Modern Roundabout

In 1966, the concept of the modern roundabout was created in Great Britain [4][2]. In modern roundabouts, instead of using nearside priority, which is standard in conventional intersections and at traffic circles, vehicles entering the roundabout yielded to vehicles already circulating in the roadway. The new priority rules attempt to prevent the circulatory roadway from gridlocking, as excess traffic queues up on the approaches and not in the intersection itself [5]. Additionally, other features of the modern roundabout include, yield controlled approaches and approach deflection, which serve to slow entering vehicles [2]. Figure 1 shows several important characteristics of modern roundabouts.

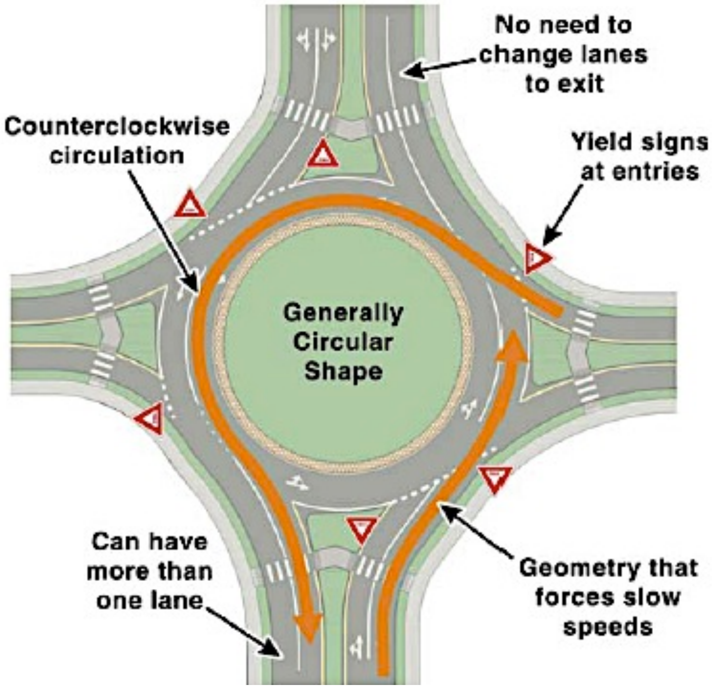

Figure 1. Characteristics of Modern Roundabouts [2]

The design improvements of the modern roundabout have made roundabouts a safe and operationally feasible alternative to conventional stop controlled or signalized intersections. Roundabouts have been used for decades in other parts of the world as a successful traffic control measure [6]. Modern roundabouts have been used in Great Britain since the 1960's and in France since the 1970's. Additionally, many other countries such as Sweden, Norway, South Africa, Australia, and others have built modern roundabouts [7].

1.1.2 Types of Modern Roundabouts

There are three different types of roundabouts: mini, single-lane, and multi-lane. The Georgia Department of Transportation (GDOT) design policy manual specifically discusses single and multi-lane roundabouts and refers the user to NCHRP Report 672 for guidelines on mini-roundabouts [8]. The basic geometric characteristics of the modern roundabout are shown in Figure 2. Table 1 shows some of the differences between the three types of roundabouts.

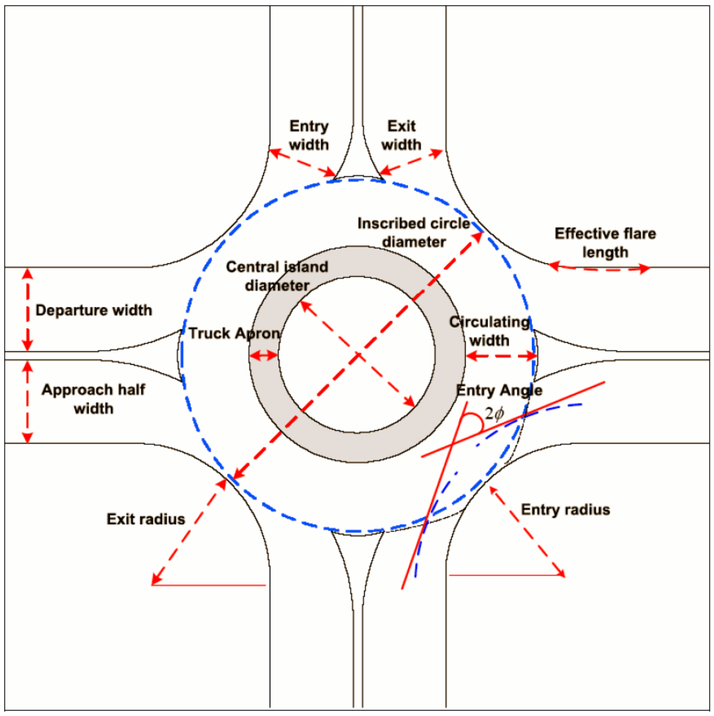

Figure 2. Geometric characteristics of modern roundabouts [9]

Table 1. Characteristics of Roundabouts [2]

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	15 to 20 mph (25 to 30 km/h)	20 to 25 mph (30 to 40 km/h)	25 to 30 mph (40 to 50 km/h)
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	45 to 90 ft (13 to 27 m)	90 to 180 ft (27 to 55 m)	150 to 300 ft (46 to 91 m)
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout

*Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.

The primary difference between the different types of roundabouts is size. Mini-roundabouts are the smallest type of roundabouts with inscribed circle diameters generally between 45 to 90 feet. The central island is another distinguishing feature of mini roundabouts. The central island for a mini-roundabout is fully mountable and they usually have only one circulating lane in the circular roadway [2]. There are few mini-roundabouts in the United States. The first mini-roundabout to be built in the United States was built in Dimondale, Michigan in 2001 [10].

Single-lane roundabouts are the most common roundabout type in the United States. Single lane roundabouts have inscribed circle diameters of approximately 90 feet to 180 feet. Similar to mini-roundabouts, single-lane roundabouts also have only one lane in the circular roadway but they are able to accommodate more traffic than mini-roundabouts. The central island for a single-lane roundabout is different from a mini-roundabout in that it is not fully mountable. Passenger cars and buses will usually be able to travel within the circular roadway and there may be a truck apron to accommodate vehicles with larger turning radii [2].

The third type of roundabout is the multi-lane roundabout. Multi-lane roundabouts have more than one lane on some portion of the circular roadway. The number of lanes does not have to remain constant within the circular roadway. For example, the approaches on the major road may have two lanes while the approaches on the minor street have only one. Multi-lane roundabouts are generally larger than single-lane roundabouts with inscribed diameters of 150 feet to 300 feet [2].

1.1.3 Roundabouts in the United States

Despite the success that modern roundabouts have had in other parts of the world, the construction of modern roundabouts in the United States has been relatively slow paced. Roundabouts were not commonly built in the United States until the 1990's. The first roundabout to be proposed in the Unites States was for Ojai, California [5]. However, public backlash against the project kept the roundabout from being built. The first roundabouts to be built in the United States were constructed in 1990 in Summerlin, Nevada [5].

By 2000, the Federal Highway Administration published the document, "Roundabouts: An Informational Guide" that gives guidance on the design, operation, and maintenance of roundabouts [11]. Following this publication, the National Cooperative Highway Research Program conducted a study, NCHRP 3-65, on roundabouts that included capacity, delay, speed, and crash records. This study led to the publication of the NCHRP report 572 and later NCHRP report 672. NCHRP 672 is the second edition of the FHWA's "Roundabouts: An Informational Guide" [2]. The results from the NCHRP study were presented at the first International Roundabout Conference in 2005 [9].

Roundabouts have gained much support in recent years within the United States. According to the FHWA [12]:

> *Roundabouts are the preferred safety alternative for a wide range of intersections. Although they may not be appropriate in all circumstances, they should be considered as an alternative for all proposed new intersections on federally-funded highway projects particularly those with major road volumes less than 90 percent of the total entering volume. Roundabouts should also be considered for all existing intersections that have been identified as needing major safety or operational improvements. This would include freeway interchange ramp terminals and rural intersections.*

At the time that the NCHRP study was conducted, 310 roundabouts were identified in the United States [13].

Familiarity with the roundabout varies from state to state and city to city. Several states such as: Kansas, Maryland, New York, Wisconsin as well as others have created state roundabout programs [14]. In Carmel, Indiana alone there are over 60 existing roundabouts [1]. However, despite the prevalence of roundabouts in some states and areas of the country, roundabouts are still relatively uncommon in some states.

1.2 Safety Benefits

Roundabouts provide many benefits in terms of safety for both motorists and pedestrians as compared with traditional intersections. At a roundabout there is a reduction in the severity of accidents as compared with traditional intersections designs [2]. One way in which accident reduction is achieved is by the reduced number of conflict points. A modern roundabout with four legs only has eight conflict points, while a traditional intersection has 32 conflict points. Figure 3 is an illustration of the conflict points at a traditional intersection of two streets and a modern roundabout with four legs. As can be seen from Figure 3, the merging and diverging conflicts are reduced by half at a modern roundabout while the crossing conflicts are eliminated completely [11]. According to a study undertaken by the Institutes for Highway Safety it was found that a single-lane roundabout can decrease the number of crashes by 61 percent and 58 percent in urban and rural areas respectively over using stop-control and by 32 percent over using a traffic signal. An even greater reduction in injury crashes was noted at single-lane roundabouts [15].

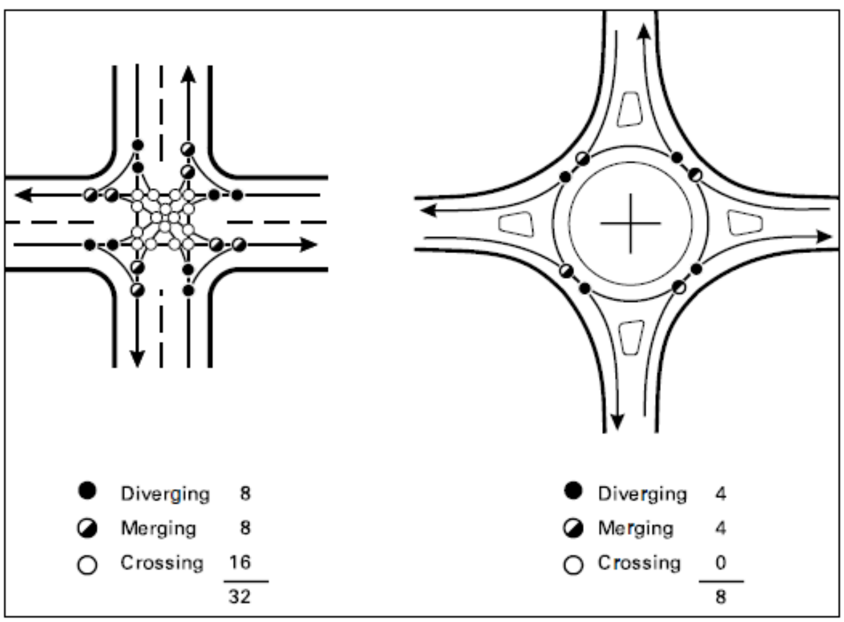

Figure 3. Conflict points at intersections [11]

Roundabouts are also considered safer for bicyclists than traditional intersections. Bicyclists are allowed to use the circular roadway as a vehicle or dismount and use the crosswalks as a pedestrian. The physical geometry of the roundabout slows vehicles to speeds that are more similar to that of a bicycle making the intersection safer for cyclists who traverse it as a vehicle [2].

Similarly, roundabouts can also be a safer design for pedestrians. The presence of a splitter island creates a pedestrian refuge. Thus, pedestrians can cross in two stages, which allow them to only have to check one direction of traffic at a time before crossing[13]. Also, the slower speeds reduce the chance of pedestrian fatality should an accident occur. However, even though the roundabout is considered a safer design for pedestrians in general, the continuous flow nature of the roundabout has created some concern about the safety of visually impaired pedestrians at crossing the street at roundabouts. There are treatments that can be used to increase the safety at roundabouts for these pedestrians such as pedestrian signals and surface treatments [16].

1.3 Purpose and Need

The purpose of this project is to calibrate the HCM 2010 roundabout capacity equations to reflect the characteristics of Georgia drivers. There were no roundabouts in Georgia included in the NCHRP study on roundabouts. Therefore, the current HCM 2010 roundabout capacity equations may not accurately predict capacity at roundabouts in Georgia. Calibrating these equations will allow for better estimates of roundabout capacity. FHWA recognizes that while there are many positive safety aspects of roundabouts, they are not appropriate for all instances [12]. Therefore, being able to accurately predict capacity will allow for more informed decision making for a particular setting.

In order to calibrate these equations for Georgia, several tasks were undertaken. First, all of the known roundabouts in Georgia were identified. Then a subset of this list was chosen for data collection based on a variety of factors including location and geometric characteristics. For the chosen roundabouts, field data collection, which consisted of using video cameras to capture roundabout operations in the field, was completed. The videos were then processed in the laboratory to extract the necessary data to calibrate the HCM 2010 roundabout capacity equations.

This project focuses on single lane roundabouts. However, calibrating the equation for single lane roundabouts will allow for the realistic determination of whether single lane roundabout will operate well in the field or whether a multilane roundabout should be considered. Therefore, this project promotes safety by increasing the amount of knowledge available when a decision is being made between a single line and a multilane roundabout.

CHAPTER 2

LITERATURE REVIEW

In order to calibrate the HCM 2010 roundabout capacity equations, it is necessary to identify the current state of practice for roundabouts in Georgia. Also, the data required to calibrate these equations must be identified. Past studies to calibrate these equations were identified as well as the current practice in roundabout data collection practices.

2.1 Roundabouts in Georgia

In Georgia, roundabouts have been built by the Georgia Department of Transportation (GDOT) as well as other entities such as counties, cities, and private developers [17]. FHWA has identified roundabouts as a way to improve safety. Additionally, GDOT also prefers roundabouts for some locations because; “GDOT also considers roundabouts as the preferred safety alternative for a wide range of intersections on public roads” [8]. Chapter eight of the GDOT Design Policy Manual is dedicated to roundabouts [8].

The GDOT Design Policy Manual outlines the steps for conducting a roundabout feasibility study and guidance on when they are necessary. As part of feasibility studies, an operational analysis is performed. GDOT provides the GDOT Roundabout Analysis Tool in the form of a spreadsheet for analyzing the performance of roundabouts [8]. The GDOT tool provides analysis for both single-lane and multi-lane roundabout and yields results using both a calibrated and uncalibrated version of the HCM 2010 model. Despite the fact that the GDOT calculator uses a calibrated HCM 2010 model, the values are calibrated using data from California and the City of Bend Oregon and have not been calibrated for Georgia conditions or Georgia drivers [18].

Roundabouts in Georgia are found in a variety of rural, urban and suburban settings. One of the earliest roundabouts to be built in Georgia was built in Carroll County in the city of Whitesburg in 2000. Since then, roundabouts have been built in many other counties in Georgia including Coweta, Dawson, DeKalb, Fulton, Cobb, Glynn, and others. Additionally, there are many other roundabouts that are currently in various stages of planning, design, or construction around the state of Georgia. Most of the roundabouts in Georgia are single-lane roundabouts with either three or four legs. However, there are several multi-lane roundabouts in Cherokee County as well as one in Glynn County on St. Simon's Island[17].

2.2 NCHRP Studies

NCHRP 3-65 is a study that took place from 2002-2004 to gather data from roundabouts in the United States. The results of this study were presented at the first International Roundabout Conference in 2005. Also, the results of this study are included in NCHRP 572 and NCHRP 672 [9].

In this study, a list of all known roundabouts in the United States was compiled and data was gathered on many aspects of roundabouts such as capacity, delay, speeds, and safety. However, only 31 roundabouts from a ten states were used for data collection. The states in which data was collected are: Nevada, Colorado, Maryland, Vermont, Maine, Michigan, Kansas, Utah, Washington, and Oregon. In some states, such as Colorado and Washington, several roundabouts were used for data collection. To choose roundabouts for data collection, the first criterion listed by the authors is, "The likelihood of finding continuous (persistent) queuing on one or more of the roundabout approaches, representing capacity conditions” [9]. Additionally, the team considered the locations of the roundabouts and strove to choose both single-lane and multilane roundabouts with various geometric characteristics. [9].

Mast-mounted video cameras were used to collect data at the sites. According to NCHRP 3-65 the equipment used for recording of the roundabout operations included masts, digital cameras, omni-directional cameras, and DVD recorders [9]. The data collection team itself consisted of four people.

NCHRP Report 572, presents the data and results that were gathered as part of NCHRP 3-65 [13]. Roundabout data from the collection sites was used in existing capacity equations to determine how well the existing equations estimate capacity at American roundabouts based on the observed data. The models that were compared are models from the United Kingdom, Australia, Germany, France, Switzerland, the HCM 2000, and the FHWA. For almost every single-lane roundabout, the uncalibrated equations predicted higher capacities than were observed and similar results were observed for multi-lane roundabouts for every model except the HCM 2000 model [13]. Figure 4 shows a graph of the uncalibrated equations and the raw data.

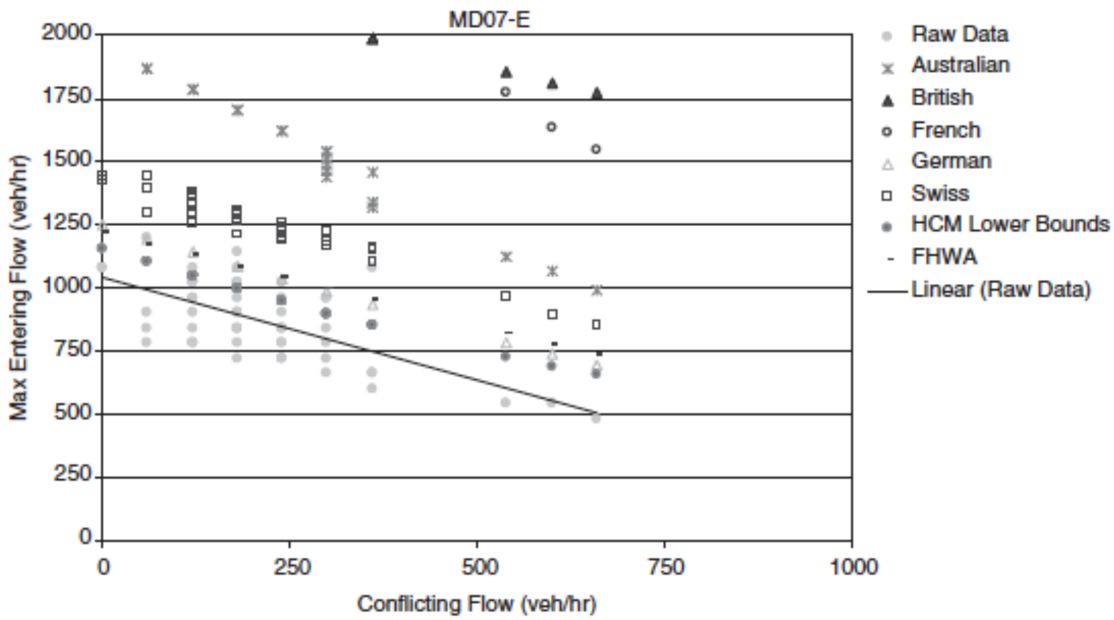

Figure 4. Uncalibrated roundabout capacity equations and field data at a single-lane roundabout

The calibrated models were found to be more accurate in predicting observed capacity than the uncalibrated models. In the report, the methodology for finding critical and follow-up headway is discussed and used in the appropriate models along with other relevant parameters for calibration. A recommended capacity model is developed for single-lane and multi-lane roundabouts and these are the capacity models that are included in HCM 2010. These capacity models are also presented in NCHRP 672, which is the second edition of FHWA's "Roundabouts: An Informational Guide" [2]. These models are presented in the next section.

2.3 HCM 2010

Chapter 21 of the Highway Capacity Manual 2010 is dedicated to roundabouts. The capacity models and methods that are outlined in HCM 2010 come from the data and results found in NCHRP 572 [3].

There are three different capacity equations that are presented in HCM 2010 that are used for five different configurations of approach lanes and circulating lanes. The first equation given is the capacity for single-lane roundabouts, in which each approach is single-lane and there is a single-lane circular roadway [3]. The equation is shown below as Equation 1. Figure 5 shows a graph of this equation with field data. Both exponential and linear regressions were done on the raw data. However, the exponential equation was ultimately selected [13].

$$c_{e,pce} = 1{,}130e^{(-1.0\times 10^{-3})v_{c,pce}} \quad (1)$$

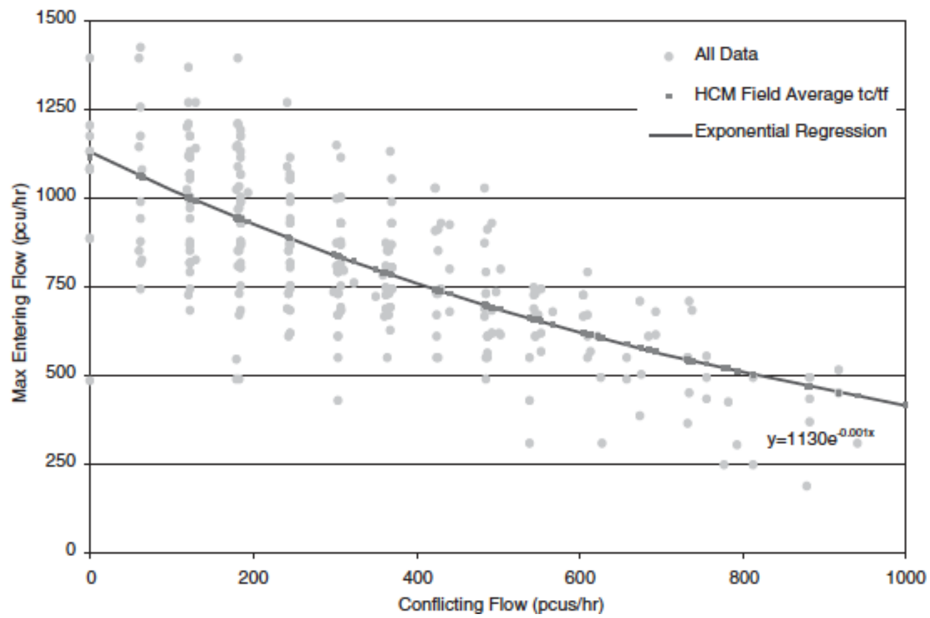

Figure 5. HCM 2010 Single-lane capacity equation with field data [13]

In this equation, $c_{e,pce}$ is the capacity of the lane under consideration where the volume is given in passenger car units. $v_{c,pce}$ is the conflicting flow rate also given in equivalent passenger car units. Figure 6 shows the traffic flows at a single-lane roundabout. This is the same equation that is found in NCHRP Report 572. In addition to the single-lane equation, HCM 2010 also provides an equation for the configuration where there is a single lane circular roadway and two lanes on an approach. This equation is the same as the equation for the single-lane roundabouts shown above [3].

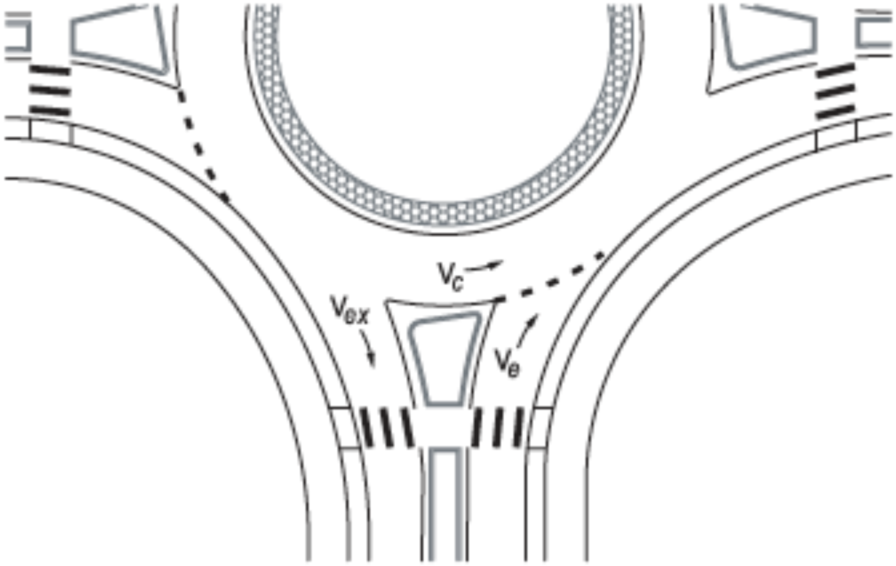

Figure 6. Circulating flow, entering flow, and exiting flow at a single-lane roundabout approach [2]

The HCM 2010 also provides capacity equations for multilane roundabouts, which in HCM 2010 are configurations with two lanes in the circular roadway. The HCM 2010 does not provide capacity for multi-lane roundabouts that have three or more circulating lanes [3]. For the configuration where there are two circulating lanes and one approach lane, the capacity of the approach lane is determined by Equation 2 [3]. Figure 7 shows a graph of this capacity equation.

$$c_{e,pce} = 1{,}130e^{(-0.7\times10^{-3})v_{c,pce}} \qquad (2)$$

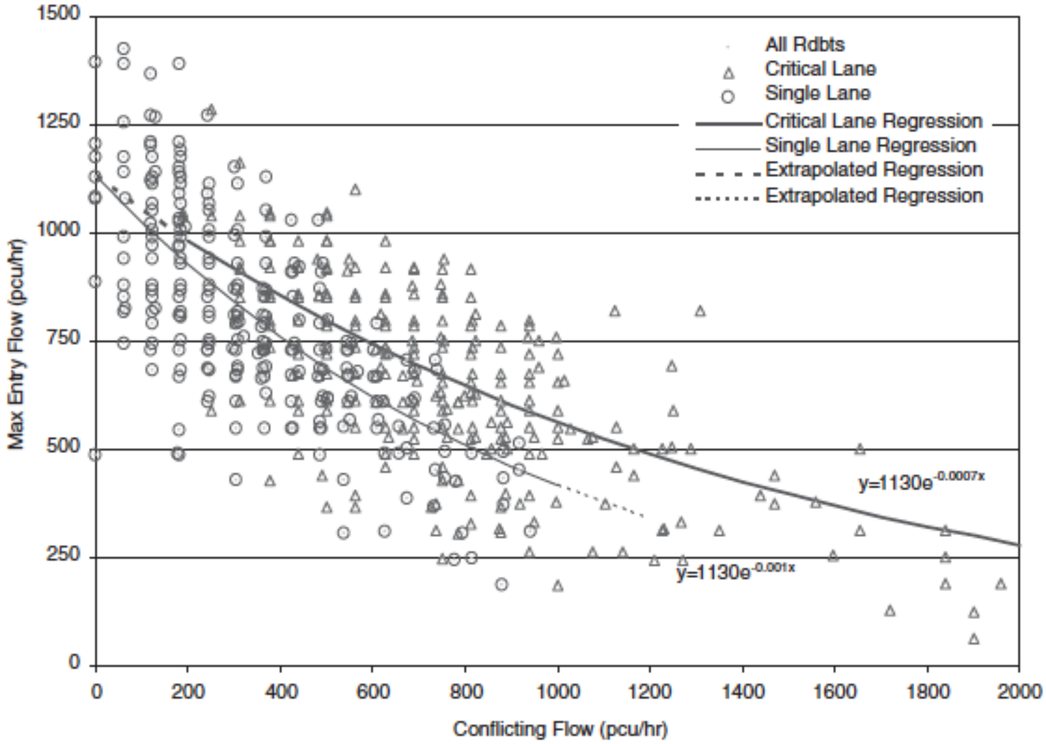

Figure 7. NCHRP capacity equations with raw data

There are two equations provided for an approach that has two lanes feeding into a two lane circulating roadway. The first equation is for the right lane, which is considered the critical lane according to NCHRP 572 because it is, "the most heavily utilized lane" [13].The right lane equation is the same as Equation 2. Equation 3 is for the left lane of a two-lane approach [3].

$$c_{e,pce} = 1{,}130e^{(-0.75\times10^{-3})v_{c,pce}} \qquad (3)$$

HCM 2010 also provides equations for slip lanes. Figure 8 shows an example of a slip lane. There are two equations given. The first is for a single slip lane with a single exiting lane and the second is for one slip lane against two exiting lanes. The first case is covered by Equation 1 for single lane roundabouts and the capacity under the second case is found using Equation 2, or the equation for the critical lane of an approach for a multi-lane roundabout. There are no capacity equations given for non-yielding slip lanes, due to lack of US data; however they are mentioned and assumed to have high capacity [3].

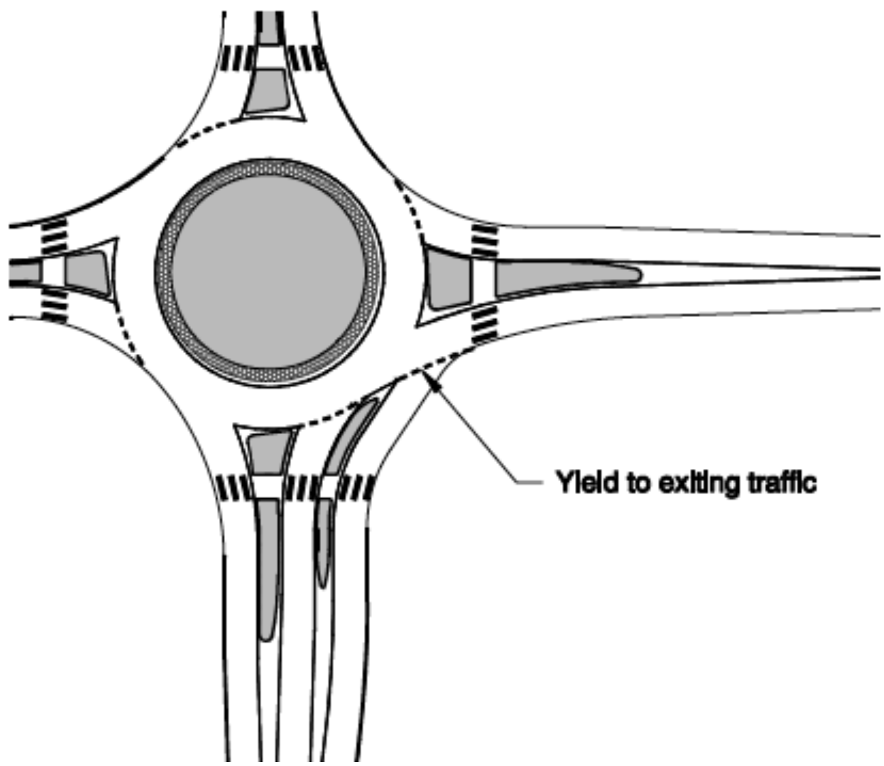

Figure 8. Example of a yielding slip lane [2]

As is seen from the equations presented in HCM 2010, the HCM 2010 model provides capacity per lane rather than per approach. According to Akçelik, these roundabout models are "unique in HCM 2010 in the sense that HCM models for other intersection types are by lane groups" [19]. However, for single lane roundabouts with single lane approaches, a lane and an approach are the same.

As seen, the roundabout capacity equations in HCM 2010 only require the conflicting flow rate as an input. However, the general forms of the equations are given to allow for calibration of the capacity equations to local conditions [3]. These equations are designed so that they can be calibrated and stay current with the changes in the data [20]. The general equation takes the form of Equation 4, where the parameters in the equation are described by Equation 5 and Equation 6 [3].

$$C_{pce} = Ae^{-Bv_c} \qquad (4)$$

$$A = \frac{3{,}600}{t_f} \qquad (5)$$

$$B = \frac{t_c - \frac{t_f}{2}}{3{,}600} \qquad (6)$$

In these equations, t_f is the follow-up headway and t_c is the critical headway. According to NCHRP 572, at a roundabout, critical headway is defined as "the minimum headway an entering driver would find acceptable" [13]. Critical headway cannot be measured in the field because drivers will accept all gaps larger than their critical headway [13]. However, it can be estimated by measuring the lengths of gaps in the circulating stream that are either accepted or rejected by entering vehicles. Additionally, lags measured at the roundabout can also be used in the calculation of critical headway. A lag is the time between when a vehicle arrives at the entrance point and the next circulating vehicle. According to NCHRP 572, a lag is simply a portion of a larger gap [13]. If exiting vehicles are included in the analysis, then they are also used along with circulating vehicles to calculate gaps and lags in the conflicting flow. Follow-up headway is defined as “the headway maintained by two consecutive entering vehicles using the same gap in the conflicting stream” [13]. Thus, if two vehicles enter the roundabout from the approach with no conflicting event between them, a measure of follow-up headway can be made. Figure 8 and Figure 9 show illustrations of critical and follow-up headway respectively.

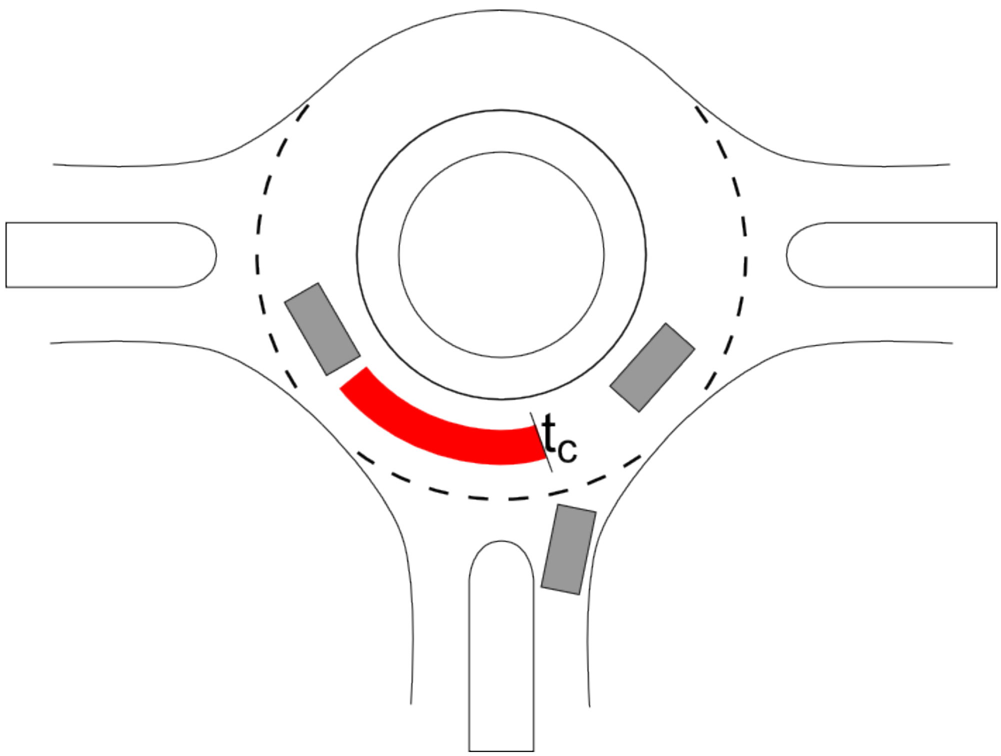

Figure 9. Critical Headway (Diagram by Aaron T. Greenwood)

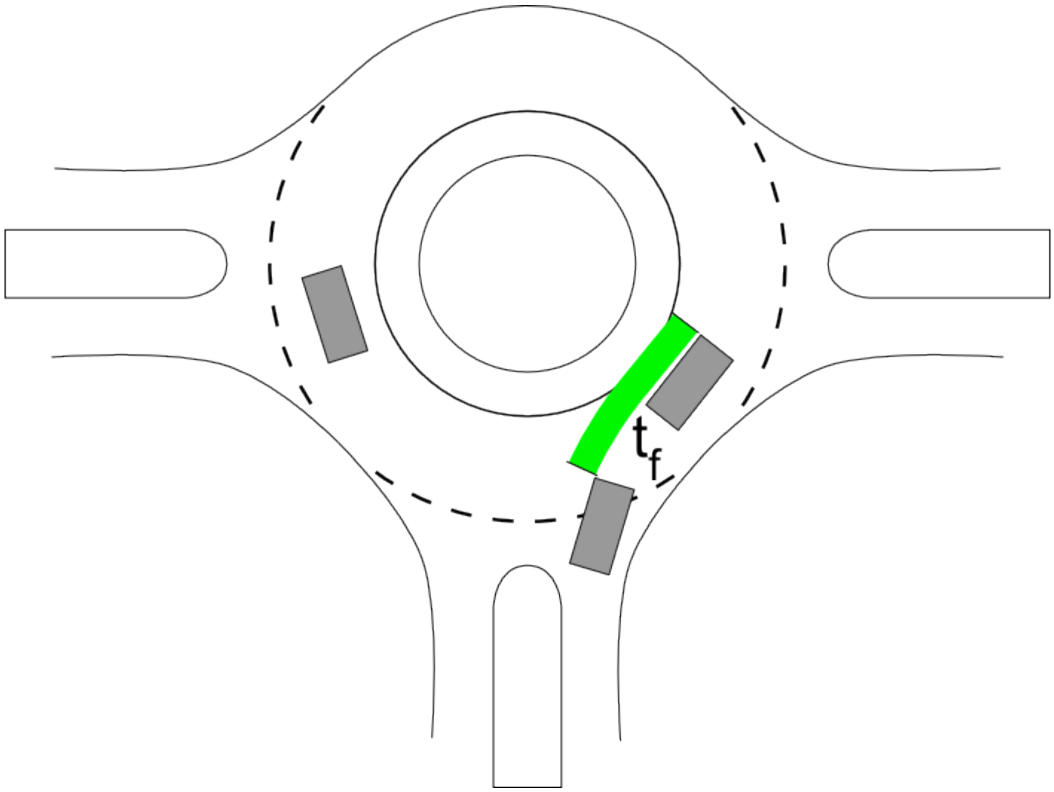

Figure 10. Follow-up Headway (Diagram by Aaron T. Greenwood)

Even though the HCM capacity equations are the HCM procedure for calculating the capacity of roundabouts in the United States, the HCM recognizes that these equations have limitations and, in certain situations, using other means for determining capacity may be advisable. For instance, roundabouts that have unusually high volumes of pedestrians and bicycles and/or signals to accommodate these users could be modeled with other methods. Also, multilane roundabouts that have three or more lanes in the circulating roadway are not covered in the HCM equations and thus another analysis method would be needed to analyze a roundabout with this geometry [3].

2.4 Data Required for Calibration

From Equation 3, Equation 4, and Equation 5 it is seen that follow-up headway and critical headway are required to calibrate the HCM 2010 capacity equations to local driving conditions. In order to collect the headway data, roundabout operations are recorded in the field and a computer program is used to record timestamps. Figure 11 from NCHRP 572 shows where the timestamps are collected at the roundabout.

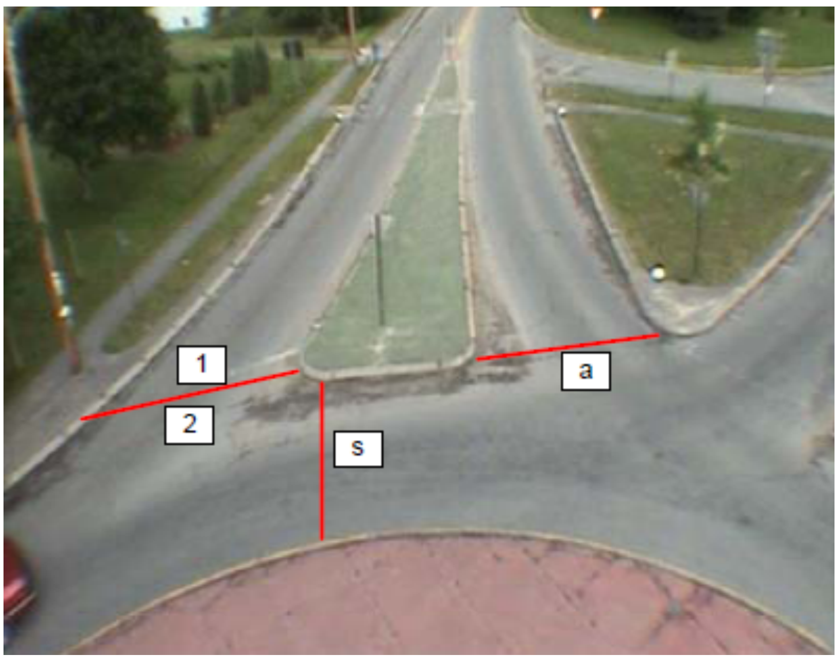

Figure 11. Physical location of timestamps [21]

In the current HCM 2010 method, the timestamps used to determine capacity are the timestamps collected at positions 1, 2, and s in Figure 11. The line corresponding with "a" is the exit time of a vehicle on the circular roadway. However, exiting vehicles were included in the final capacity equation in NCHRP 572 [13]. The use of these timestamps to determine follow-up headway and gap data is discussed in the following sections.

2.4.1 Follow-up Headway

Follow-up headway can be determined directly from operations observed in at the roundabout. To determine follow-up headway, timestamps collected at “1” and “2” in Figure 11 are used. Multiple vehicles must enter the roundabout during the same gap for a measurement of follow-up headway to be taken. Queuing must be present on the approach for an accurate calculation of follow-up headway [13].

2.4.2 Critical Headway

Critical headway, the other measurement required for calibration of the capacity equations is more difficult to obtain than follow-up headway. Critical headway cannot be observed and must be estimated from the accepted and rejected gaps. To determine the duration of accepted and rejected gaps, the difference between subsequent timestamps collected at line “s” in Figure 11 is calculated [13]. At lease one study found that timestamps collected for exiting vehicles affect the critical gap [22]. However, exiting vehicles were not included in the final NCHRP 572/HCM 2010 models [13].

Additionally, NCHRP 572 presents three different methods for determining critical gap. These methods are [13]:

> *(1) inclusion of all observations of gap acceptance, including accepted lags;*
> *(2) inclusion of only observations that contain a rejected gap; and*
> *(3) inclusion of only observations where queuing was observed during the entire minute and the driver rejected a gap.*

The average critical headway found by the NCHRP 572 research team using Method 1 is 4.5 seconds. When Method 2 was used, the average critical headway increased to 5.0 seconds and Method 3 yielded an average critical headway of 5.1 seconds. However, there was not enough data at some of the roundabouts to calculate the critical headway using Method 3. Additionally, Method 1 had the added complication of including lags. In order to include gaps and lags in the data, the researchers for the NCHRP study estimated gaps from the lags by adding a few seconds of headway time to the lag [13]. However, A study of roundabouts in Wisconsin also used lags to calculate critical headway but no mention is made of converting lags to gaps [22]. The authors recommend Method 2 for determining critical headway, which does not include lags [13].

Since critical headway cannot be field measured, the gaps and lags that are measured in the field are used to estimate the critical headway. The maximum likelihood method is used in NCHRP 572 and in many other studies as well to determine the critical headway [13]. Troutbeck compared the maximum likelihood method to several different methods and found that the maximum likelihood method has low bias compared to some of the other methods such as the Ashworth method or the Ramsey and Routledge method [23]. Troutbeck developed a code in FORTRAN to implement the maximum likelihood method [23]. The maximum likelihood method was used for this study and will be covered in more detail in Chapter 3.

2.5 Calibration Efforts/Case Studies

2.5.1 CALTRANS

One of the first calibrations of the NCHRP 572 capacity equations was performed for the California Department of Transportation (Caltrans). Caltrans' *Roundabout Geometric Design Guidance,* which includes the calibrated capacity equations, was published in June of 2007 [6]. This study found values for critical headway and follow-up headway to calibrate the capacity equation for single-lane roundabouts as well as for the right and left lanes of multi-lane roundabouts [6].

For the Caltrans calibration effort data was collected at seven single-lane and three multi-lane roundabouts. Two hours of video was collected at each roundabout using a tripod-mounted camera positioned to capture the traffic on the most heavily utilized leg of the roundabout. Videos were made between either 7:00am -9:00am, 11:30am-1:30pm, or between 4:00pm-6:00pm for each roundabout. Four hours of data were collected at several of the roundabouts and a total of 26 hours of data was recorded [6].

Following the collection of field data, the data from the videos were analyzed using *TDIP (Traffic Data Input Program)*. Similar to the NCHRP study, critical headway and follow-up headway were found by recording timestamp position of the vehicles as indicated in Figure 11. According to the *Roundabout Geometric Design Guidance,* "Three time events involving an entering vehicle were recorded: the time when an entering vehicle stopped at the entrance line, the passage times of circulatory vehicles that directly conflicted with the entering vehicle, and the time at which the stopped vehicle passed the entrance line. The passage times of circulating vehicles defined the start and end of major stream headways that were either accepted or rejected by the entering vehicles" [6]. An Excel program was then used to change these values into the necessary headway data to obtain the critical headway and the follow-up headway. The follow-up headway was determined directly from the data gathered and the Maximum Likelihood Methodology was used to find the critical headway. Critical and follow-up headway were found for both single-lane roundabouts as well as the right and left lanes of multi-lane roundabouts [6].

For single lane roundabouts it was found that the critical headway that should be used for California is 4.8 seconds while the follow-up headway should be 2.5 seconds. For multi-lane roundabouts the critical and follow-up headways determined were a little lower. For the right lane of a multilane roundabout the critical headway should be 4.7 seconds while for the left lane the critical headway used should be 4.4 seconds. Follow-up headway for both the right and left lanes of a multilane roundabout was found to be 2.2 seconds [6]. The resulting calibrated capacity equation for single lane roundabouts in California is shown as Equation 7 below.

$$c = 1{,}440e^{-0.0010 \times v_c} \qquad (7)$$

The calibrated capacity equations for multilane roundabouts are shown below as Equation 8 and Equation 9 for the right and left lanes respectively.

$$c = 1{,}640e^{-0.0009 \times v_c} \qquad (8)$$

$$c = 1{,}640e^{-0.0010 \times v_c} \qquad (9)$$

In addition to finding California specific values for follow-up headway and critical headway, comparisons were also made with headway values from existing roundabout capacity models. Table 2 shows the comparison between the headway values.

Table 2. Headway comparison table [6]

Model		Critical Headway (seconds)		Follow-up Headway (seconds)	
		One lane	Two lane	One lane	Two lane
HCM		4.1 to 4.6	N/A	2.6 to 3.1	N/A
Germany[1]		4.4	4.4	3.2	3.2
France[1]		N/A	N/A	2.1	2.1
NCHRP 3-65	Left lane	4.2 to 5.9 (5.1)[2]	4.2 - 5.5 (4.5)	2.6 - 4.3 (3.2)	3.1 - 4.7 (3.4)
	Right lane		3.4 - 4.9 (4.2)		2.7 - 4.4 (3.1)
California	Left lane	4.5 - 5.3 (4.8)	4.4 - 5.1 (4.7)	2.3 - 2.8 (2.5)	1.8 - 2.7 (2.2)
	Right lane		4.0 - 4.8 (4.4)		2.1 -2.3 (2.2)

Notes: 1. Results obtained from NCHRP Report 572 (7)
2. Numbers in ( ) indicate the average value

Note: In this table HCM refers to HCM 2000

As can be seen in Table 2, the critical headway values determined as part of this study for California are similar to those found in other models. The average critical headway for single lane roundabouts is within the range of values given in the NCHRP 3-65 though the average for NCHRP 3-65 is slightly higher. The critical headway value for California is slightly higher than those found from the HCM or German models. Additionally, the values for critical headway for the right and left lanes of multi-lane roundabouts are similar to those found in NCHRP 3-65 and the German model [6].

2.5.2 City of Bend Oregon

The City of Bend Oregon has also calibrated the roundabout capacity equations to local driving conditions. These values are presented in the *City of Bend Roundabout Operational Analysis Guidelines,* prepared for the city by Kittelson & Associates, Inc in 2009 [24]. The capacity model that is presented in the document is the NCHRP 572 equation with a critical headway of 4.1 seconds and 3.2 seconds for the follow-up headway. These values are for single-lane roundabouts [24]. The calibrated values result in Equation 10.

$$c_{e,pce} = 1{,}130e^{(-0.0007)v_{c,pce}} \qquad (10)$$

This equation is for single-lane roundabouts only. There is also a capacity equation given for multi-lane roundabouts but it is the same as the equation that is presented in NCHRP Report 572 [24].

For the single-lane roundabouts, the critical headway is just out of the 4.2-5.9 range of critical headway values that was observed in the NCHRP report 572 and it is well below the national average of 5.1 seconds. By contrast the 3.2 seconds identified for the City of Bend is within the 2.6 to 4.3 range identified for follow-up headway and is also the same as the national average observed [24].

2.5.3 Case Study of Anchorage, Alaska

A case study to calibrate the NCHRP equation for multi-lane roundabout was performed in Alaska. This case study involved data collection at one roundabout in Anchorage, Alaska. A picture of this roundabout is shown in Figure 12.

Figure 12. Roundabout in Anchorage, Alaska (Source: Google Maps, accessed 6/28/2012)

Though there are two roundabouts, one on either side of New Seward Highway, only the roundabout on the west side was used in this case study. This roundabout includes pedestrian sidewalks and right turn slip lanes [25].

Data was collected at this roundabout using tripod-mounted video cameras. The data collected was used to find the critical headway and follow up headway for this roundabout. These values were then used to calibrate the HCM 2010 model for multi-lane roundabouts. The critical headway at this roundabout was found to be 4.28 seconds and the average follow-up headway was found to be 2.58 seconds [25]. These two values yield Equation 11.

$$\boldsymbol{c_{crit}} = \mathbf{1,395}e^{-0.0008v_c} \qquad (11)$$

It was expected that the calibrated equation would provide higher capacity estimates than the HCM 2010 model and upon comparison this was found to be the case for conflicting vehicle flows up to 2100 pcu/hr [25].

2.5.4 Wisconsin

Wisconsin has more than 200 modern roundabouts in operation [22]. The Wisconsin Traffic Operations and Safety Laboratory at the university of Wisconsin-Madison conducted a study of capacity at Wisconsin roundabouts. For this study fourteen different roundabouts were studied with four of these roundabouts used for the capacity study. Of the fourteen roundabouts twelve of them are multilane roundabouts and two are single-lane roundabouts. For the capacity study two single-lane and two multi-lane roundabouts were studied. However, five approaches were studied as two approaches on one of the multilane roundabouts were used. Several roundabouts were also used for a speed study [22].

For the capacity study, video data was collected using the Miovision™ camera system and HD video cameras. Cameras were placed upstream and downstream of the approach as well as on the perimeter of the roundabout. Since three different cameras were used to record operations at each roundabout the videos were synchronized before the data was extracted [22].

Timestamps were collected from the videos for use in determining the follow-up and critical gap. Follow-up headway was only considered under saturated conditions, which were determined by the research team to be, “a state when at least one vehicle has been waiting behind before the leading vehicle entered the roundabout” [22]. The average follow-up headway found that the single-lane roundabouts were 2.6 seconds and 3.8

seconds. At the multi-lane roundabouts the follow-up headway was found to be 3.0, 2.8, and 2.4 seconds [22].

Critical headway was determined using the maximum likelihood method, which is the method also used in NCHRP 572 [22]. A program was developed to implement this method. It was found that the critical headway at the single lane roundabouts is 5.5 seconds and 4.8 seconds. For the multilane roundabouts the critical headway was found for the right and left lanes separately. For the right lane of the multilane roundabouts the critical headways were found to be 3.4 seconds, 3.8 seconds, and 4.4 seconds. For the left lane the critical headways on the studied approaches were found to be 4.1 seconds, 4.2 seconds, and 4.8 seconds [22].

2.5.1 Carmel, Indiana

A study was performed to create capacity equations for several roundabouts in Carmel, Indiana. In addition to creating local capacity equations, the NCHRP capacity equations were also calibrated to local conditions for comparison with the new capacity models. The study was presented at the 2011 Transportation Research Board meeting as well as at the 3rd International Conference on Roundabouts [1].

The authors suggest the development of localized capacity models instead of just using local calibration of the HCM 2010 equations because according to the authors there is no way to be sure that once the model is calibrated that it will necessarily present an adequate representation of actual conditions. Also, since the NCHRP 572/HCM 2010 model was developed with data from only a few states, the equation may not work well in other states even when calibrated. According to the study authors “underestimating or overestimating roundabout capacity can have significant implications for decision makers" [1]. Underestimating capacity can lead to the creation of premature congestion and overestimating capacity can lead to selection of another intersection type when a roundabout would in practice perform adequately [1].

For this study, data was collected in one-minute periods at roundabouts that experienced congestion or queuing in at least fifteen periods per hour were used. Ultimately three roundabouts were chosen for the study and data collection times were determined by during which times of the day congestion was present at the roundabouts. Data collection was performed using the Miovision™ Video Collection Unit to record operations, which is shown in Figure 13. In the Miovision™ unit, a video camera is attached to top of a telescoping pole to record traffic operations. Video data collection was used due to the difficulty of manual data collection at roundabouts and all legs of each roundabout were recorded during the same data collection period [1].

Figure 13. Miovision™ data collection unit [1]

The second part of the process involves data extraction. To create their models, the authors counted the entering and conflicting flows at each roundabout recorded on the videos and only data collected during congested periods was used. For this study, data extraction was performed using two different methods. One method used was to use the Miovision™ data extraction program, which requires uploading the recorded video for automated extraction. The other method used was manual data analysis, which is significantly more time consuming for the research team but is considered essential for at least some of the data to provide a basis of comparison to the automatically generated results. The need for manually extracted data led to the creation of two data sets for this study know as “unverified data” and “verified data” with the “verified data” being the data that was manually extracted from the video [1]. Comparison showed the automatically extracted data had a high rate of accuracy most of the time and Figure 14 shows a graphical comparison of the two data sets [1].

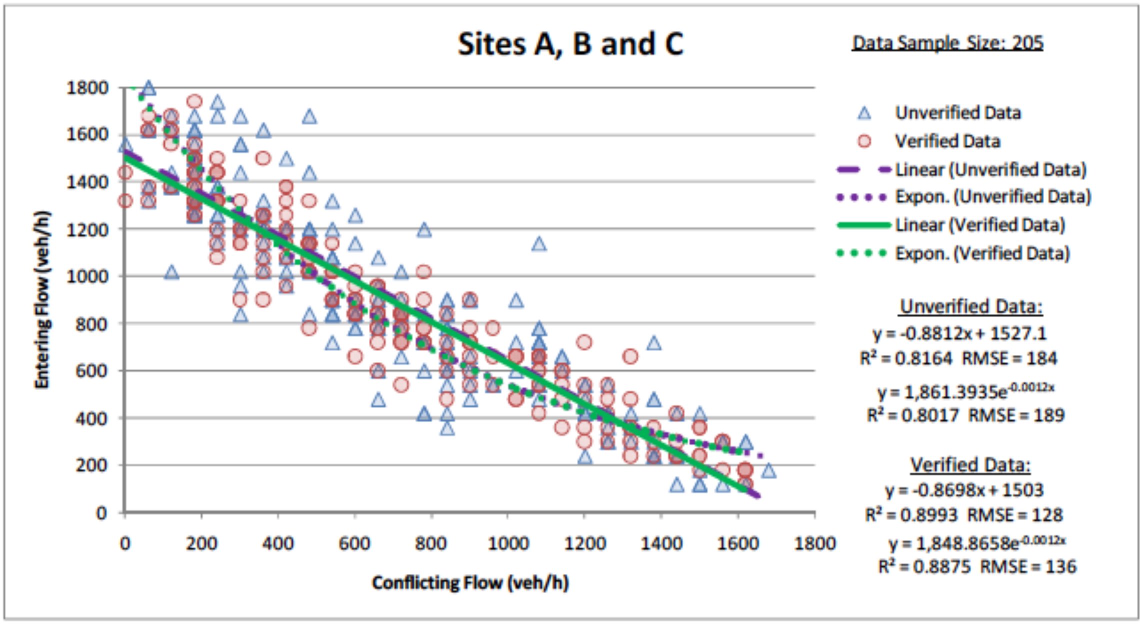

Figure 14. Comparison between verified and unverified data

The last step of the process was creating the model. Similar to NCHRP 572, each data point is a one-minute flow rate and both linear and exponential regressions were applied to the data. After a comparison the R squared values, it was determined that linear regression was a better fit for the data collected in this study. The capacity model developed in this study was also contrasted with the NCHRP model and it was found that the study’s model consistently provided higher estimates of capacity. Figure 15 shows the difference between the NCHRP model and the model developed in this case study [1].

Critical headway and follow-up headway were extracted from the data and found to be lower than the NCHRP values and thus to provide higher estimates of capacity than the uncalibrated equations.

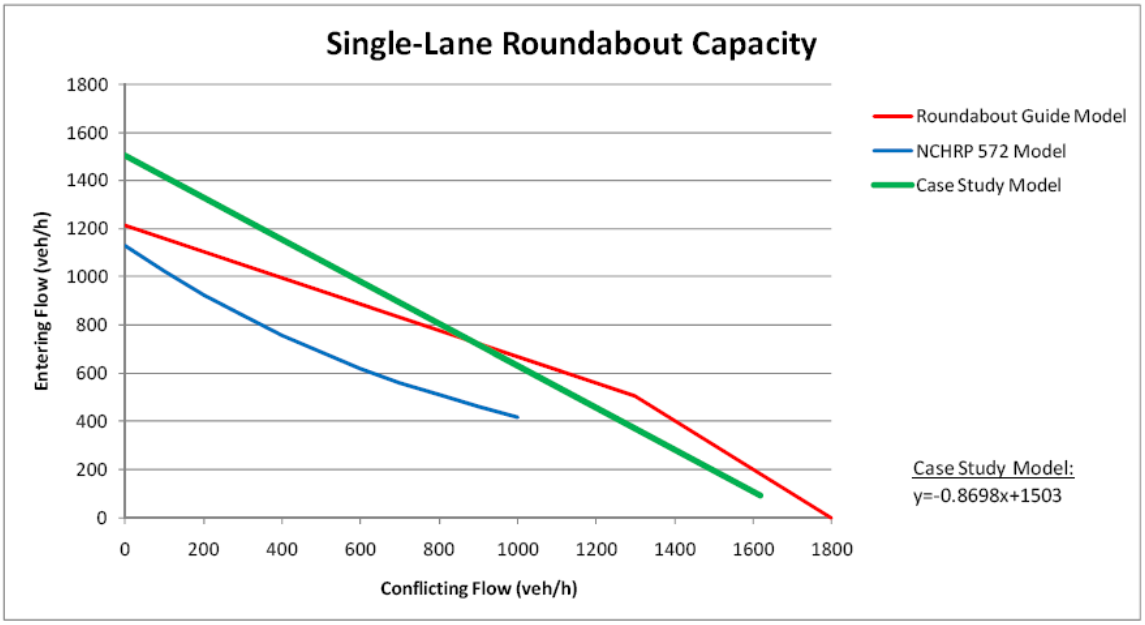

Figure 15. Comparison of capacity equations [1]

2.6 Other Factors

2.5.4 Effect of Exiting Vehicles

In NCHRP 572, an analysis of the effects of exiting vehicles was performed, but exiting vehicles were not accounted for in the final model. The Caltrans calibration study only considers circulating vehicles, remaining silent on the consideration of exiting vehicles. However, the Wisconsin study did evaluate the effect of exiting vehicles and it was determined in this study that the inclusion of exiting vehicles does impact the critical headway and follow-up headway at both single-lane and multilane roundabouts.

The researchers in Wisconsin model capacity with and without the inclusion of exiting vehicles. When exiting vehicles were included in the analysis, the gap was calculated by adding a time adjustment equal to the amount of time that it would have taken for an exiting vehicle to reach the conflict point as if it had been a circulating vehicle. Figure 16 shows the distance corresponding to the correction time. This method is known as equivalent travel time. The adjustment was added to the calculation of the gap when the second vehicle in the gap was an exiting vehicle. The adjustment was used in the calculation of both gaps and lags. It was found that exiting vehicles do affect the gap values. It was found for both follow-up headway and critical headway that the gaps are shorter when exiting vehicles are included [22]. Figure 17 and Figure 18 show the differences that were found in the critical headway and the follow-up headway when exiting vehicles were included. For every roundabout approach studied, it is seen that there is a decrease in the follow-up headway and the critical headway when the existing vehicles are accounted for [22].

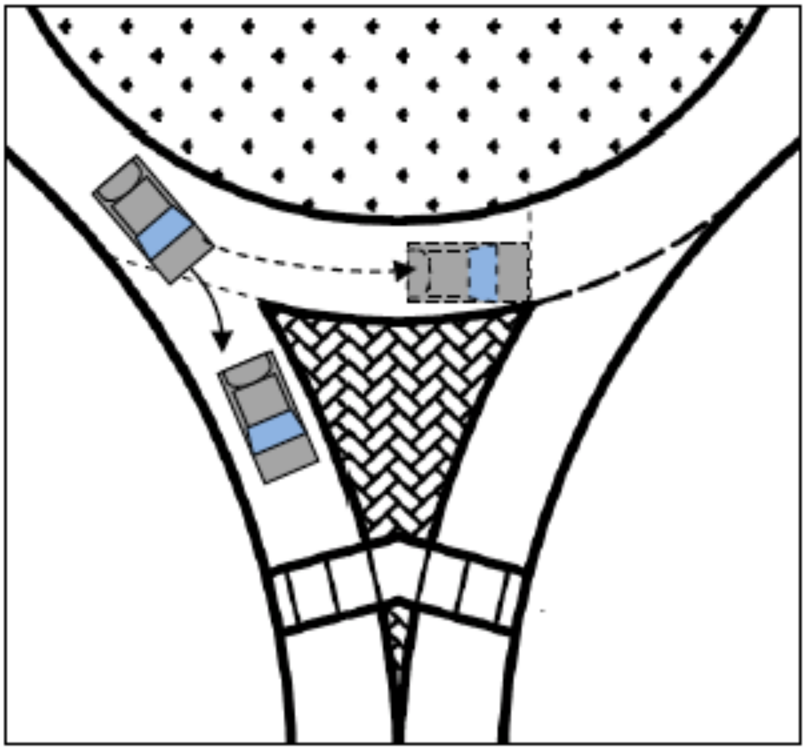

Figure 16. Wisconsin method for including exiting vehicles

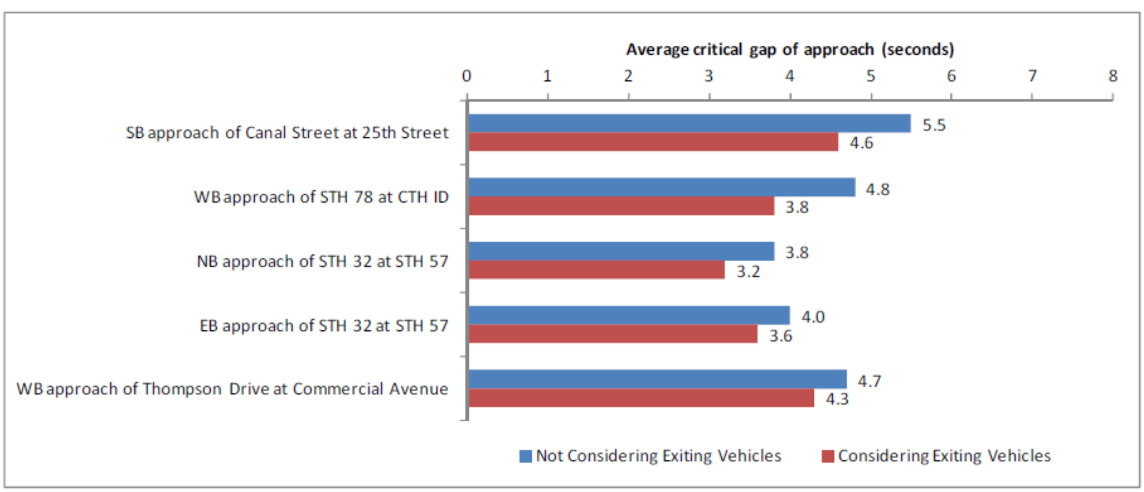

Figure 17. Effect of exiting vehicles on critical gap [22]

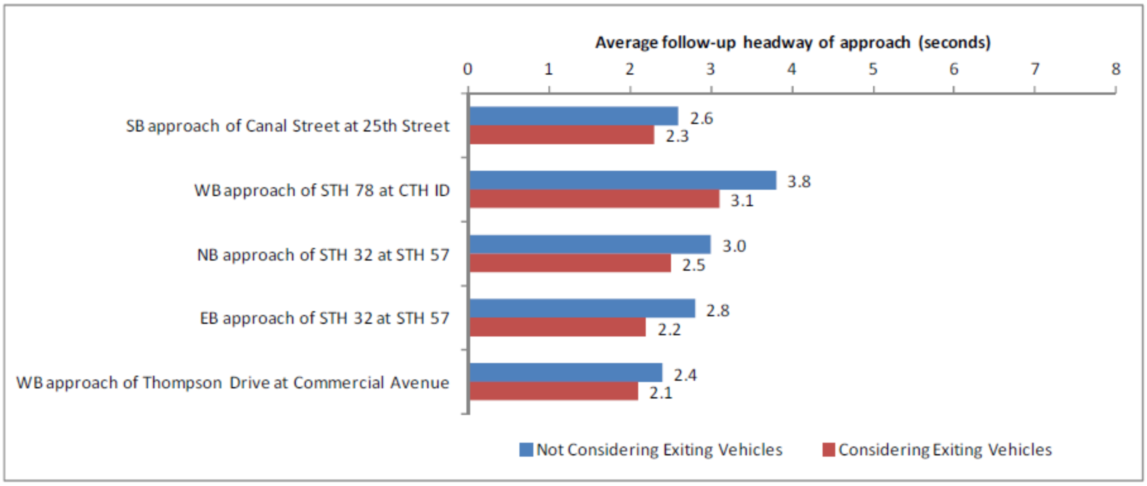

Figure 18. Effect of exiting vehicles on critical gap [22]

2.5.1 Effects of Trucks/Large Vehicles

The percentage of trucks and large vehicles is an additional factor that affects roundabout operations. The capacity at roundabouts with a large percentage of heavy vehicles may be affected because these vehicles have slower start-up times and different acceleration characteristics than standard passenger cars. A couple of studies have been completed that look at the effect of heavy vehicles on the follow-up headway and critical headway at roundabouts.

2.5.1.1Wisconsin

In addition to studying the effect of exiting vehicles, the researchers in Wisconsin also compared critical headway and follow-up headway based on vehicle type. Three vehicle types were considered: passenger cars, motorcycles, and heavy trucks. It was found that the critical headway and follow-up headway of passenger cars did not differ significantly from the headway values that were found when vehicle type was not considered. The researchers postulate that this is because motorcycles and heavy trucks were not as abundant in the traffic stream as passenger cars. The researchers considered both the case where exiting vehicles are excluded and the case where exiting vehicles are considered in the analysis. It was found that in both cases that critical headway and follow-up headway are larger for trucks than for passenger cars and lower for motorcycles than for passenger cars [22].

2.5.1.2 Brattleboro, Vermont

Another study to determine the effect of heavy vehicles on roundabout capacity was conducted at a roundabout on Putney Road and Chesterfield Road in Brattleboro, Vermont. The video footage used in this study was part of the NCHRP 3-65 data collection. The authors stated that converting trucks into passenger car units (pcu) as required by the HCM 2010 roundabout capacity equations does not adequately reflect the effect that trucks have on roundabout capacity [26].

The roundabout that was used as part of this study is a single-lane roundabout with four legs. **Figure 19** shows a picture of this roundabout. This roundabout features a truck apron in the circulatory roadway and approximately 10-15% of the traffic using this roundabout is from truck traffic.

Figure 19. Roundabout Brattleboro, Vermont (source: Google Earth™, accessed 6/28/2012).

Six hours of video data from this roundabout was used to determine the critical headway and the follow-up headway. To determine exactly how trucks affect the follow up headway, four different cases of follow up headway were identified. These follow-up cases are: “1) Car followed by Car (car/car), 2) Car followed by Truck (car/truck), 3) Truck followed by Car (truck/car), and 4) Truck followed by Truck (truck/truck)” [26]. Additionally, critical headway was determined separately for cars and trucks [26].

Critical headway was determined by graphing the curve of accepted gaps against the curve of rejected gaps, with the intersection of these curves being the critical headway as well as by using the probability equilibrium method. The critical headway for trucks was found to be 5.3 seconds using both methods. The critical headway for cars was found to be 3.8 seconds for cars using the graphing method and 3.9 seconds using the probability equilibrium method. This data shows that there is more than a second difference between car and truck headway values, clearly indicating that the amount of truck traffic at a roundabout can affect the capacity [26].

In addition to critical headway, the four cases of follow-up headway were evaluated. It was found that the smallest follow-up headway is between cars. The average of the headway for a car following a car was found to be 2.1 seconds. The next lowest follow-up headway was from cars following trucks and this headway was found to be 4.1 seconds. The longest follow-up headways were from trucks following cars and trucks following trucks. These values are 5.3 seconds and 8.5 seconds respectively. However, the highest number of observations came from the car/car case. There was only one truck/truck case. An average follow-up time was also determined from all follow-up observations involving a truck. This follow-up time was found to be 5.2 seconds, which is significantly longer than the 2.1 second follow-up time found for the case only involving cars [26].

From the differences between the follow-up and critical headways found for cars and trucks separately, it is found that trucks have an effect on capacity. The results of this study future underscore the importance of calibrating these equations to local conditions. For example, an area with many trucks will have significantly less capacity in roundabouts than an area that has a very small volume of truck traffic. The authors of this study conclude by recommending that, individual values of follow-up and critical headway should be determined for cars and trucks [26].

2.7 Field Data Collection Methods

The current primary method for roundabout data collection is video cameras to record operations in the field with video processing (typically manual) in the lab to extract the required timestamp information. Timestamps are then post processed to determine the critical and follow-up headways. The NCHRP study was performed in the early 2000s and therefore, DVD recorders were used along with the video data collection [13]. Other, more recent studies do not mention the need of such devices, presumably because of advances in portable memory or a lack of sufficient detail in the field data description. In addition, the NCHRP study used omnidirectional cameras along with standard video cameras and these cameras were placed in the central island[9]. In the Wisconsin study, several video cameras were used, and placed around the perimeter of the roundabout. These videos were then synchronized in the laboratory using a software program[22].

While the primary means for extracting data from the videos was manual observation of the video several studies, such and the study in Wisconsin and Carmel, Indiana used Miovision™ video cameras and software. As discussed earlier Miovision™ has a software application that can automatically extract the gap data from the videos recorded with Miovision™ cameras. The study in Carmel, Indiana made use of the Miovision™ program, but also did some manual data reduction to verify the results [1].

2.8 Importance of Model Calibration

From, the calibration efforts that have been completed, the benefits to calibrating the NCHRP capacity models are evident. Table 3 shows the values for critical and follow-up headway that were found as part of studies in the United States as compared to the values found in NCHRP 572.

Table 3. Critical Headway and Follow-Up Headway from different studies in the United States

<table>
<tr><th colspan="2" rowspan="2">Model</th><th colspan="2">Follow-up Headway</th><th colspan="2">Critical Headway</th></tr>
<tr><th>Single-Lane</th><th>Multilane</th><th>Single-Lane</th><th>Multilane</th></tr>
<tr><td rowspan="2">NCHRP 572</td><td>Right Lane</td><td rowspan="2">3.2</td><td>3.1</td><td rowspan="2">5.1</td><td>4.2</td></tr>
<tr><td>Left Lane</td><td>3.4</td><td>4.5</td></tr>
<tr><td rowspan="2">Caltrans</td><td>Right Lane</td><td rowspan="2">2.5</td><td>2.2</td><td rowspan="2">4.8</td><td>4.4</td></tr>
<tr><td>Left Lane</td><td>2.2</td><td>4.7</td></tr>
<tr><td colspan="2">City of Bend</td><td>3.2</td><td>--</td><td>4.1</td><td>--</td></tr>
<tr><td colspan="2">Carmel, Indiana</td><td>3.19 - 3.79</td><td>--</td><td>2.1 - 2.43</td><td>--</td></tr>
<tr><td colspan="2">Alaska</td><td>--</td><td>4.28</td><td>--</td><td>2.58</td></tr>
<tr><td rowspan="2">Wisconsin</td><td>Right Lane</td><td rowspan="2">2.6 - 3.8</td><td>2.2 - 3.0</td><td rowspan="2">4.8-5.5</td><td>3.4 - 4.4</td></tr>
<tr><td>Left Lane</td><td>2.5 – 3.1</td><td>4.1 - 4.8</td></tr>
</table>

The calibrated capacity equations that are presented by the City of Bend and by Caltrans provide capacity estimates for their respective localities that are different than those provided by the NCHRP 572 equation with no calibration. Figure 20 shows a graphical comparison between the NCHRP recommended capacity model and the calibrated models for single-lane roundabouts.

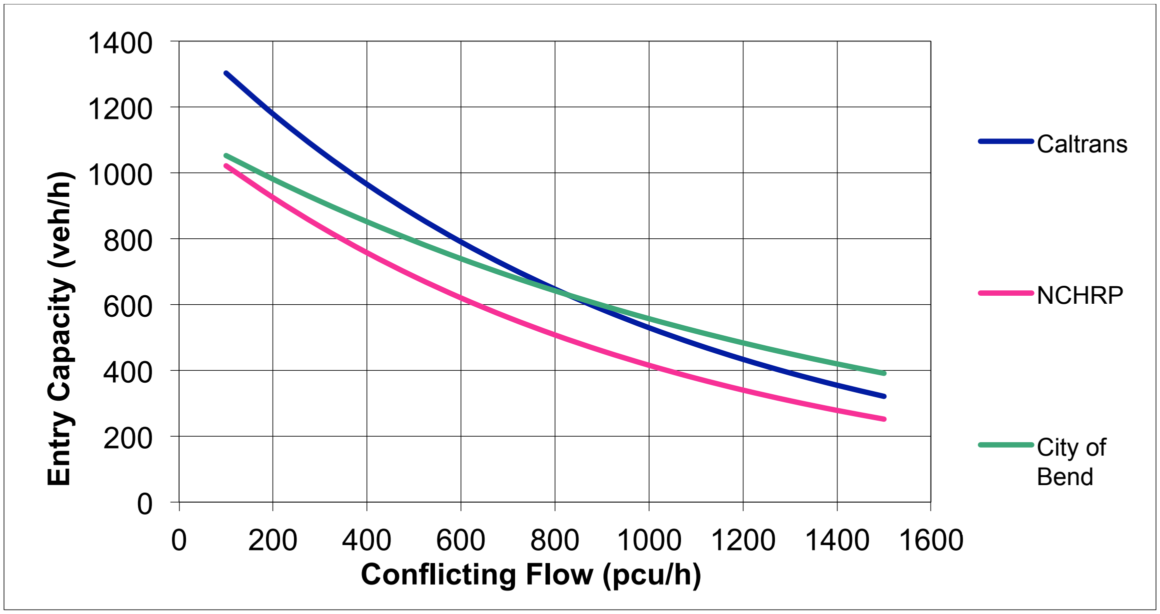

Figure 20. Calibrated capacity equations for single-lane roundabouts

As can be seen from Figure 20 the equation found by NCHRP predicts less capacity than either the Caltrans or City of Bend calibrated equations. The equations found by Caltrans and the City of Bend are similar, though the curve for the City of Bend is flatter. At approximately 800 vehicles per hour of conflicting flow, the equations yield the same capacity. At higher conflicting flows the City of Bend's equation is yields more capacity and at lower conflicting flows, the Caltrans equation yields more capacity.

Though there are clear benefits to calibration of the NCHRP multi-lane roundabout equation the effects are less pronounced than for the single-lane roundabouts. Figure 21 shows a comparison for the calibrated multi-lane roundabout equations.

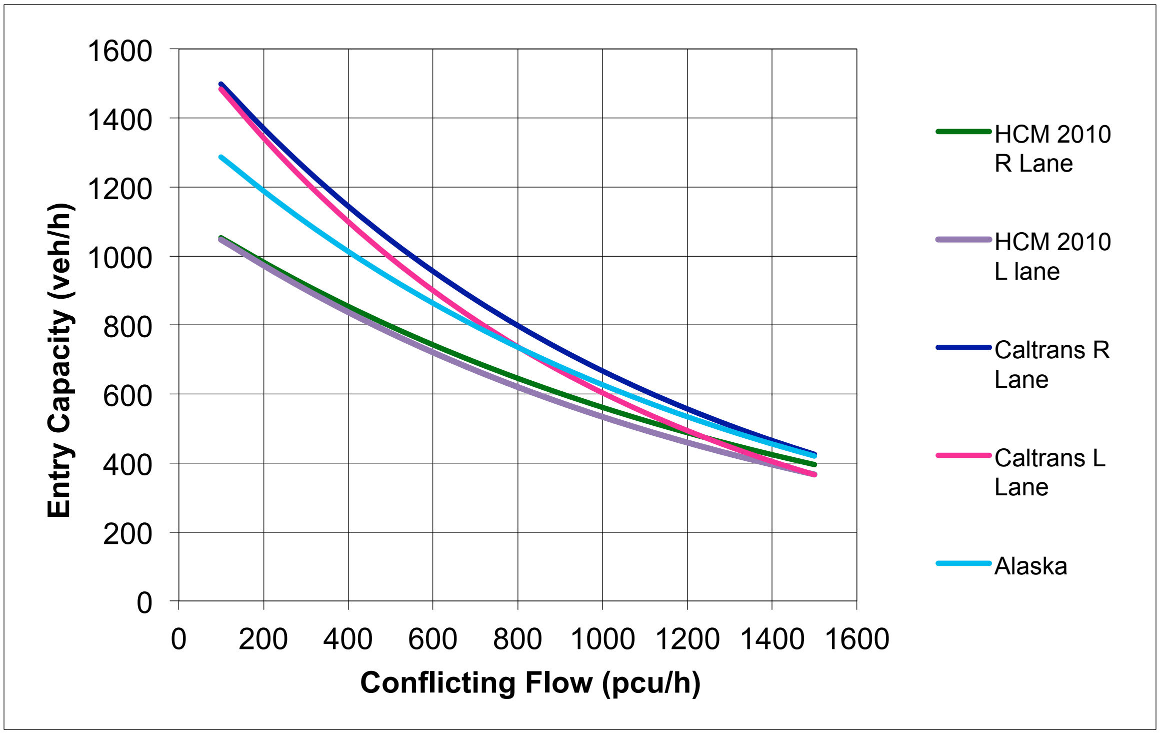

Figure 21. Calibrated multilane roundabout equations

Caltrans provides calibrated equations for multi-lane roundabouts for both the right and left lanes and both are shown in Figure 21. Both equations from Caltrans yield higher capacity estimates at low conflicting flows. Only the study from Caltrans provided more than one multi-lane capacity equation.

CHAPTER 3

METHODOLOGY

Data were collected at several roundabouts in Georgia. The roundabouts for the study were chosen based upon a variety of factors including location, traffic volume, geometry, and age. Video data were recorded at the selected roundabouts and post-processed to extract timestamps for events of interest to calculate the follow-up headway and the critical headway, which are needed for calibration.

3.1 Roundabout Selection

In order to select roundabouts for data collection, a list of roundabouts in Georgia was compiled. This list was compiled from existing lists of roundabouts in Georgia as well as by searching local news articles for articles about recently constructed roundabouts. It was found that there are over 100 roundabouts in Georgia; however, many of these roundabouts are in residential and other low volume locations [17].

Roundabouts were chosen for the study based on a variety of factors. The Georgia Department of Transportation (GDOT) State Traffic and Report Statistics website was used to determine roundabouts that are operating in high volume locations [27]. The website was used to obtain AADT information for the approaches for each roundabout where available. Also, the age, location, and geometry of the roundabout were taken into consideration. Roundabouts that have been open for at least one year were preferred over recently constructed roundabouts, when possible. However, the most important factor of roundabout selection is that the roundabouts exhibit the characteristics of modern roundabouts and that they have a sufficient traffic volume for data collection.

Additionally, roundabouts without splitter islands were not included on the GDOT list of modern roundabouts and therefore were not considered for this study[17]. Figure 1 shows the characteristics of modern roundabouts. The roundabouts in Covington, Roswell, and Fayetteville were all included in this study because they have these characteristics. Aerial images of these roundabouts can be seen in Chapter 4. An aerial image of a circular intersection that was not considered suitable for this study is shown in Figure 22 below. This intersection has stop control on the approaches and therefore does not meet the criteria for classification as a modern roundabout. This roundabout also does not have splitter islands. Site visits were completed for each roundabout to observe traffic and determine where cameras could be placed to capture traffic operations in the peak period.

Figure 22: Example of a traffic circle (Source: Google Earth™, accessed 6/28/2012)

3.2 Data Collection

The data collection team at each roundabout consisted of two people. The data collection equipment consists of:

1) 2 Panasonic HDC-TM700 video cameras
2) 2 tripods fitted with camera mounts
3) 1 ladder 6'-8'
4) 2 camera batteries (with 1 or 2 extra)

Two legs of each roundabout were recorded at a time. One camera was used per approach to record operations on the approach. The cameras were set up outside the perimeter of the roundabout such that the camera would be closer to the vehicles entering the roundabout on the approach of interest than to the vehicles exiting the roundabout on that leg. Figure 23 illustrates a camera set up at a roundabout in Cobb County. Aerials showing the locations of the cameras at the roundabouts during data collection along with the area recorded by each camera are given in Chapter 4.

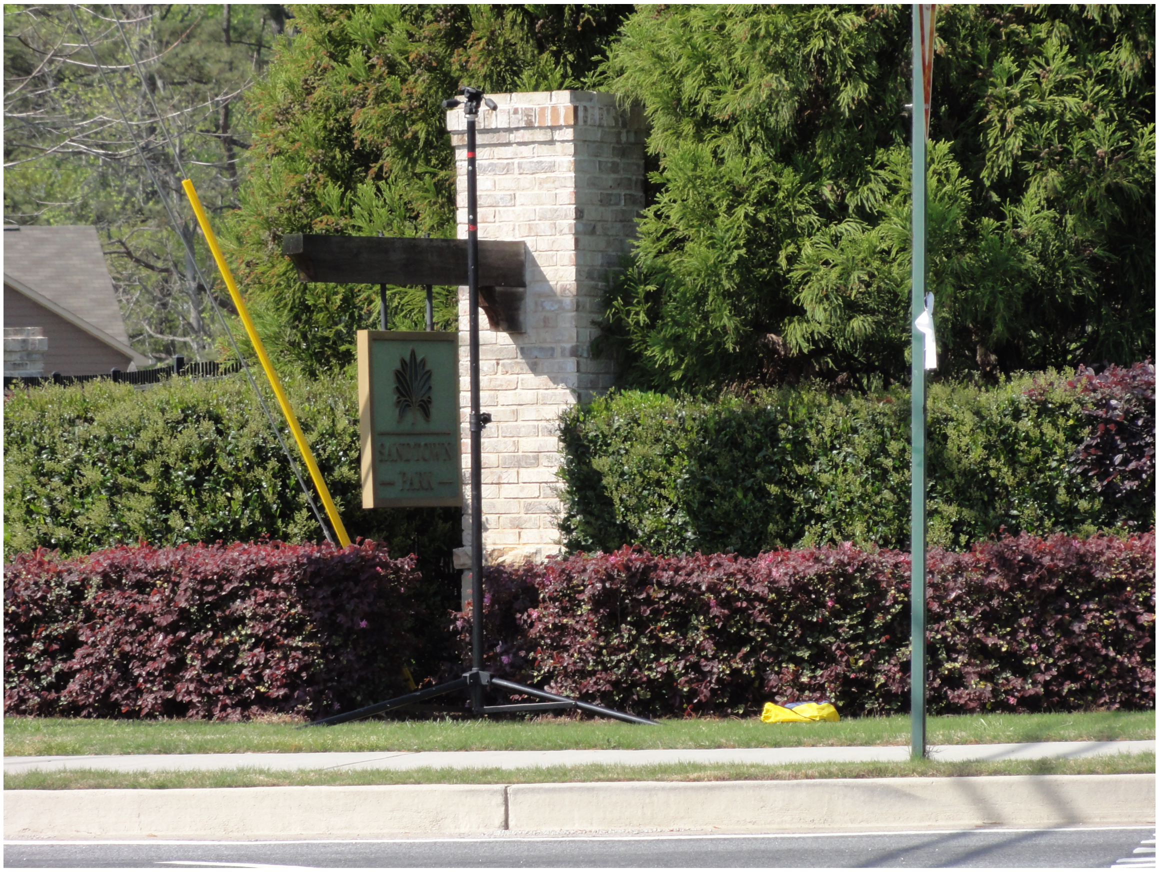

Figure 23. Typical camera setup

The angle of the camera was set in such a manner to record the entering, circulating, and exiting traffic on one leg. One camera was used per leg because at most roundabouts it was found to be difficult to capture the entering, exiting, and queued traffic on multiple legs from the same camera. Also, using only one camera per leg allowed the data collection teams use to aim and focus the video cameras to obtain the best view of the approach. Since queued traffic is required to accurately determine the follow-up headway, cameras were angled so that at least part of the queue would be visible. Figure 24 shows the view from the camera at the Fayetteville roundabout location for the southbound approach and Figure 25 shows the view from the camera at the Covington roundabout location on the southbound approach.

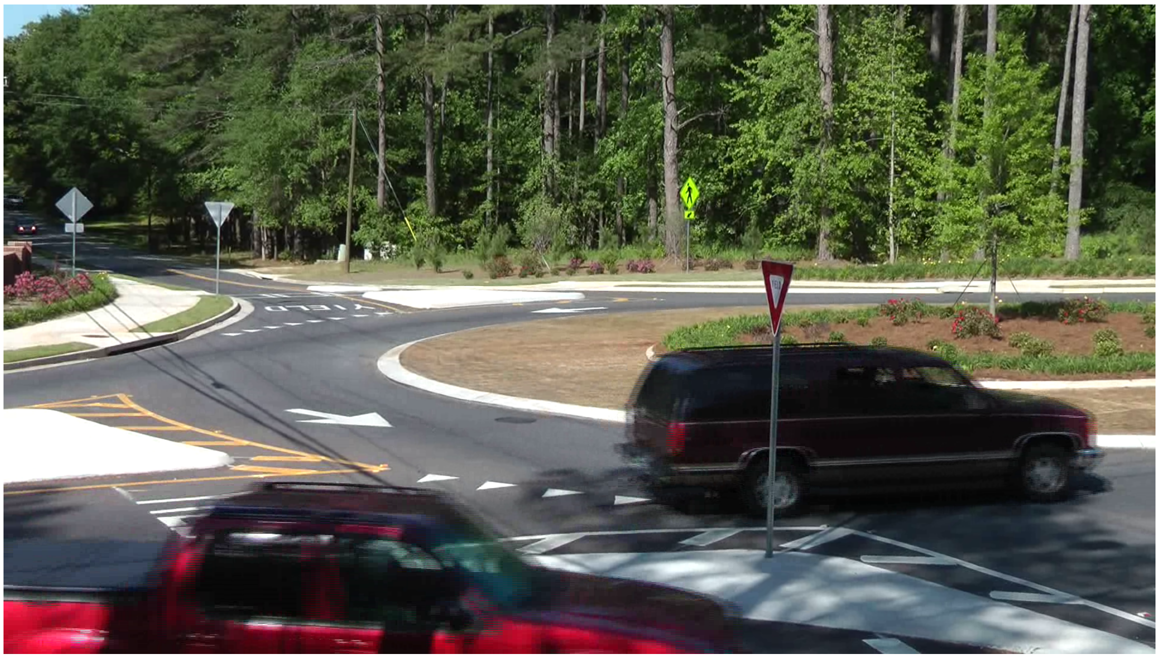

Figure 24. View from camera at Fayetteville location, looking from the southwest corner

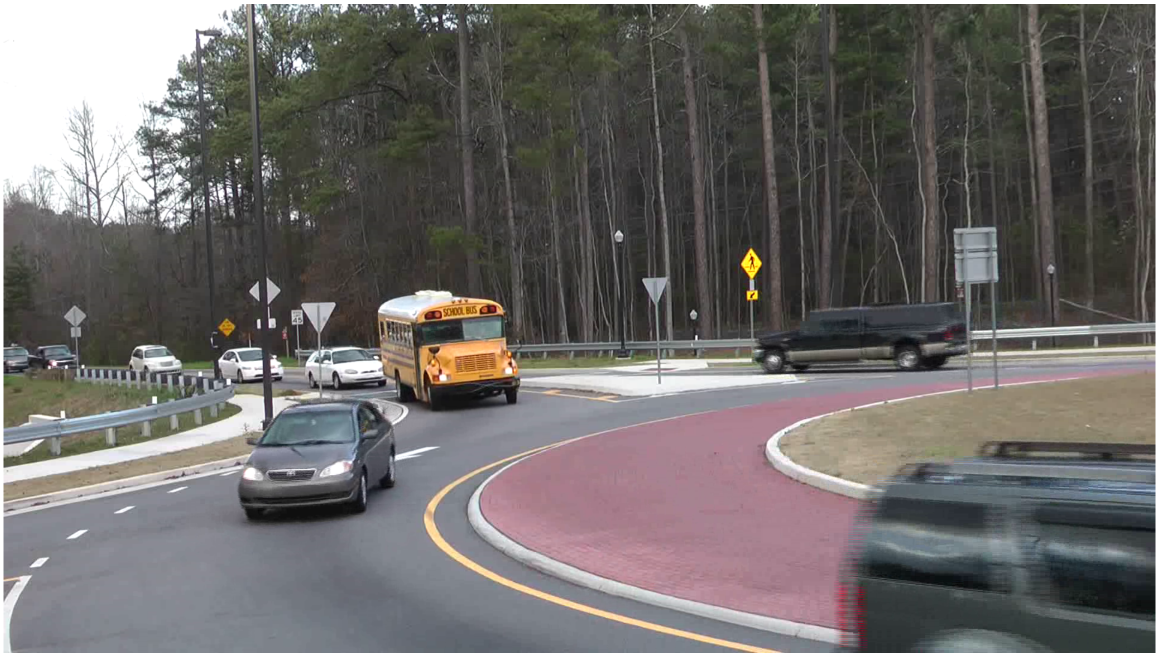

Figure 25. View from camera for Covington southbound approach, looking from the southwest corner

The cameras were set out during the PM peak period at each roundabout between about 4:30 and 7:00 PM on weekdays. As the internal memory of each camera is 32 GB, video was recorded directly onto the camera's internal memory. The camera batteries last for approximately two hours each and thus each data collection period lasted for around two hours. The data collection team set the cameras up on the tripods that are approximately 10' feet tall. The ladder was used to enable the data collection team to set the angle of each camera after placing it on the tripod. After the cameras were set up, the data collection team returned to the truck, with the ladder. The team stayed in the truck so to minimize the distraction to drivers and in many cases, the cameras and tripods were obscured due to the presence of trees, signs and other vertically oriented structures at the roundabouts. An example of how the view of the tripods was obscured by the presence of trees in the background is shown in Figure 26. The truck was parked such that the team could watch the cameras during the entire data collection period, while staying out the view of drivers in the roundabout. At the conclusion of the data collection period, the team took removed the cameras and the tripods.

Figure 26. Video camera and tripod camouflaged by trees at Covington

3.3 Data Reduction

To calculate the required follow-up headway and the critical headway at each location, timestamps were collected from the recorded videos. The video cameras recorded the videos into a mts video format. A program, FFmpeg, was used to convert the videos into an avi file format for easier analysis. In order to ensure accuracy and repeatability of the data reduction process, a video editing software, AVS Video Editor by Online Media Technologies Ltd. was used to draw lines on each video corresponding to the location of where timestamps would be collected [28]. The lines are drawn in different colors so that they contrast with the video and each other. Figure 27 shows the physical locations of the timestamps at the roundabout in Covington, Georgia.

A software program was created to assist in the data collection. This program allowed the user to watch the video and use keystrokes to collect timestamps corresponding to several events, which were written directly into a comma separated values file. The keystrokes corresponding to the events are shown next to the lines to assist in data collection. Table 4 shows a summary of the timestamps used, which are also described below. Appendix A shows the timestamps locations for every roundabout approach considered.

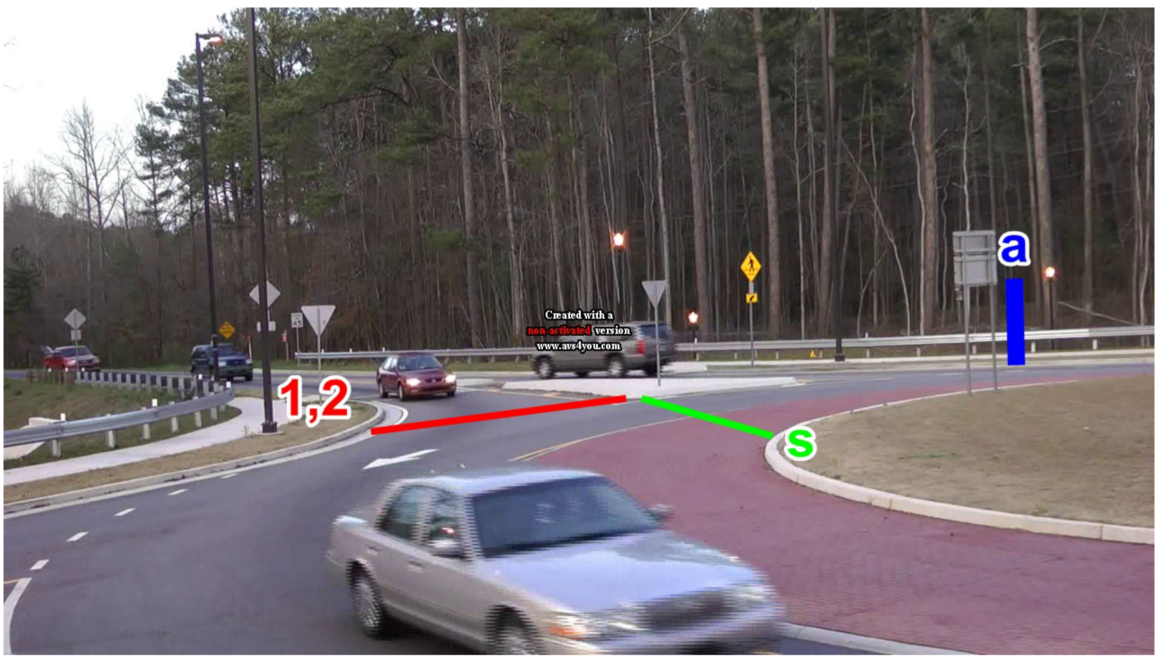

Figure 27. Timestamp locations on SB approach for roundabout in Covington, Georgia

Table 4. Summary of keystrokes

Keystroke	Event
1	Vehicle arrives at the entry point
2	Vehicle enters the circular roadway
a	Vehicle exits the roundabout
s	Vehicle circulates in front of the approach of interest
x	Beginning of queue on the approach
z	End of queuing on the approach
q	Errors in the data collection file

Keystroke “1” is for the arrival of a vehicle on the approach. This timestamp is collected when a vehicle arrives at the red line. If the vehicle did not have to stop the “1” was pressed when the vehicle reached the red line. If the vehicle had to stop, the timestamp was collected when the vehicle stops. Similarly, some vehicles reduce their speed significantly to yield to circulating vehicles without stopping. In this case, the “1” timestamp is collected when the vehicle significantly slows to yield to vehicles in the roundabout. It was noticed that many vehicles would stop before reaching the red line.

Keystroke “2” corresponds to the event where the front of queue vehicle on the approach enters the roundabout. This timestamp is collected when the front of the vehicle crosses the red line. The entry timestamp is always collected at the red line regardless of where the arrival timestamp was collected for that vehicle. For vehicles that did not stop the “2” would be pressed immediately following the “1”.

Keystrokes “s” and “a” correspond to timestamps for vehicles in the circulating roadway. Keystroke “s” is for circulating vehicles and is collected when the front of a vehicle reaches the green line. The keystroke “a” is for exiting vehicles and is collected when the front of an exiting vehicle passes the vertical blue line. A vertical line is used for exiting vehicles, instead of a horizontal line as a horizontal line would be obscured where the data point should be collected. From the camera angle shown in Figure 27, the exiting vehicles drive almost parallel to any line that would be drawn across the exit. This causes uncertainty in the exiting point location. By contrast, the lines drawn for the circulating and entering traffic (red and green) are perpendicular to the flow of traffic, improving the ability to identify the crossing point.

For the determination of follow-up headway, it was also necessary to determine time periods when there was queuing on the approach. To determine queuing timestamps “x” and “z” were used. For this analysis it was decided to require a queue of at least two vehicles long. Since a roundabout is a type of free flow intersection where vehicles are only required to stop for circulating vehicles, and in many cases vehicles do not come to a complete stop, a vehicle was considered to be in the queue when it experienced delay at the roundabout due to the vehicle in front of it. Thus, if a vehicle had to significantly slow or stop because of vehicles in front of it on the approach, then a queue was noted. The keystroke “x” was used to denote the beginning of queued conditions and the keystroke “z” was used to denote the end of queued conditions. Thus, when queuing began on the approach, a timestamps for the keystroke “x” was collected and when the last vehicle in the queue departed the approach a timestamp for the keystroke “z” was collected. Timestamps for queuing were collected as long as the queue was at least two cars long.

After converting the videos and drawing the lines, a Java program was used to manually collect the timestamps according the keystrokes above. The Java program was written by researcher Lakshmi Pessapati using Java Media Framework developed by Sun Microsystems and Fobs4JMF developed by Omnividea. To record the timestamps, the videos played back in the Java program. Then the data collector watches the videos at regular speed and presses the keystroke for the corresponding event of interest. Figure 28, shows the interface of the Java program used to collect the timestamps. The timestamps are taken directly from the video stream ensuring synchronization between multiple viewing of the video.

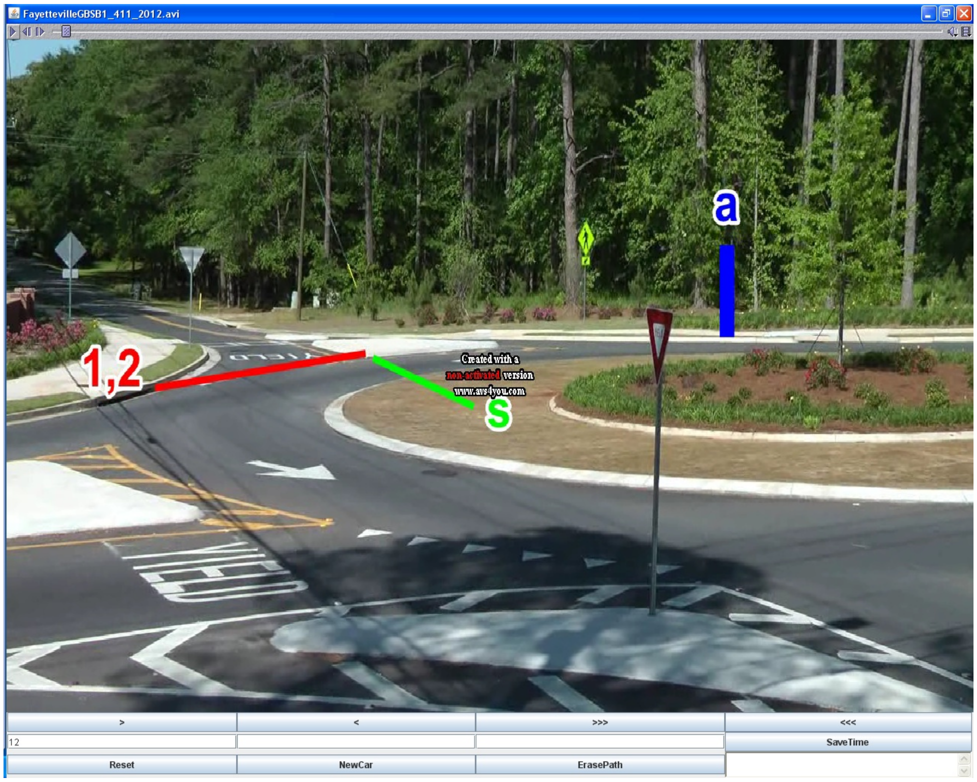

Figure 28. Screenshot of program used for collecting timestamps

Three passes through the video were used to collect the keystrokes. First, the entering and departing timestamps on the approach were collected. Then the video was viewed a second time to collect the timestamps of the circulating and exiting vehicles. The third viewing was used to determine at which points in the video there was queuing on the approach. As the timestamps are collected, the program writes them into a comma separated values (CSV) text file.

One additional keystroke "q" was used to identify errors in the data collection. If the data collector accidentally identified an exiting vehicle as a circulating vehicle, the keystroke "q" was immediately pressed to identify the error. Then, after the collector finished watching the entire video, the csv file containing the keystrokes and all of the keystrokes immediately preceding a “q” keystroke were deleted, because these are errors in the data, not actual data points. All of the timestamps corresponding to the “q” were also deleted as this keystroke is only used to identify errors and therefore is not useful for analysis. Aside from the aforementioned seven keystrokes, the Java program did not collect keystrokes from any other keys. If another key was inadvertently used, the program used a pop up window to warn the user of an invalid keystroke.

After the timestamps corresponding to arriving, departing, circulating, and exiting vehicles were collected, the data was copied into an excel file. Then, the data was sorted based on the timestamps. Formulas in excel were used to determine the follow-up headway, accepted lags, rejected lags, accepted gaps and rejected gaps. The video cameras recorded approximately 90 minutes, so it was often necessary to merge data from several videos to compile all the data for the period on the same leg into one csv file. Since the Java program only collects timestamps in reference to the beginning of the video, an extra column was added to each csv file to denote which video the data was extracted from. The data could then be copied into the same file without any confusion due to the fact that the timestamps only denote time relative to the beginning of the video.

3.4 Exiting Vehicles

Depending on the roundabout geometry, volume of exiting vehicles, and a variety of other factors, exiting vehicles may have an impact on the capacity of an approach because entering vehicles may hesitate if they cannot determine whether a vehicle is exiting or circulating. Therefore, to determine the impact of exiting vehicles at roundabouts in Georgia, two separate analyses were preformed. The first analysis excluded exiting vehicles and only used circulating vehicles to determine gaps. In this scenario, only the timestamps for the keystroke “s” are used and the timestamps for the keystroke “a” are not used. For the second case, exiting vehicles are included and both exiting and circulating vehicles were used to calculate the gaps rejected by the entering stream of vehicles. This case uses timestamps for both keystrokes “s” and “a.”

Since the exiting point is upstream of the conflict point, a method similar to that used in the study from Wisconsin, was used to calculate the perceived gaps created by exiting vehicles [22]. In this method, the distance between the exiting point and the conflicting point is estimated using the path-measuring tool to measure the distance in feet using Google Earth™. Then, the time that it would take for a vehicle to travel from the exiting point to the conflicting point is calculated using the posted speed limit. The resulting time is then added to all gaps or lags where the second car in the gap is an exiting vehicle. If the first car in the circulating roadway is an exiting vehicle, then additional time is not added because it is assumed that the driver waiting to enter the roundabout can see that the first vehicle is exiting and they are already judging the gap to the next vehicle. Adding this additional time to the gaps and lags creates the perceived gap that a driver sees, before they know whether or not the car is going to exit. The two analyses are compared to determine the effect of exiting vehicles is, if any.

3.5 Critical Headway

One of the parameters needed to calibrate the HCM 2010 roundabout capacity equations is the critical headway. The critical headway is defined by NCHRP 572 as, “the minimum headway an entering driver would find acceptable” [13]. Critical headway must be estimated from gaps that are observed in the field, as it is not possible to observe a driver's critical headway in the field. Therefore, accepted gaps and lags are used to estimate the critical headway using the maximum likelihood methodology.

3.5.1 Accepted Gap/Lag

This analysis follows method 1 in NCHRP Report 572, as presented in earlier, utilizing both gaps and lags are used to calculate the critical headway. This method was selected because some of roundabout approaches included in this study did not have enough data for analysis without including lags.   Future efforts will expand the analysis to consider the other methods.

The difference in the timestamps of the circulating and exiting vehicles, the "s" timestamps and the "a" timestamps is calculated to determine the length of the gaps and lags in the circulating stream of traffic. When exiting vehicles are excluded in the analysis, only the "s" timestamps are used and when exiting vehicles are included the "a" timestamp is used in addition to the "s" timestamp. For this study, two different analyses are performed; one that excludes exiting vehicles and one that includes exiting vehicles. The definition given in NCHRP 572 for a lag is, "the time from the arrival of the entering vehicle at the roundabout entry to the arrival of the next conflicting vehicle" [13]. A vehicle accepts the lag if upon arriving at the roundabout, it enters the roundabout in that lag time. By contrast, the lag is rejected if the arriving vehicle waits until the circulating vehicle (or exiting vehicle if exiting vehicles are being included) passes to enter the circular roadway. A gap is the time between two circulating vehicles or between a circulating vehicle and an exiting vehicle or between two exiting vehicles if exiting vehicles are being included. The gap is accepted if a vehicle waiting at the entry point enters the roundabout in the gap, and a gap is rejected if the vehicle waits at the entry point for another gap. If two or more cars enter in the same gap, then the gap is not used as a measure of gap. Rather, the measure of accepted gap used is the accepted lag, which is calculated between the last vehicle to enter in that gap and the next circulating or exiting vehicle, if exiting vehicles are being included.

For gaps to be useful, it is also necessary to determine whether or not a car accepted or rejected that gap or lag. If there are no cars on the approach for any gap in the circulating stream, then that gap was neither accepted nor rejected and is not useful for determining critical headway. To determine from the timestamps whether or not a gap was rejected, accepted or neither, the arrival (keystroke “1”) and entering (keystroke “2”) timestamps are used. For a gap/lag to be rejected, there must be one or more circulating (keystroke “s”) or exiting (keystroke “a”) timestamps before the next entering timestamp “2”. As discussed the exiting vehicle timestamp is only considered in analysis incorporating exiting vehicles. For a gap/lag to be accepted, the entry timestamp (keystroke “2”) must occur before the second circulating vehicle that is creating that gap.

3.1.1 Maximum Likelihood Methodology

The critical headway is determined by using the maximum likelihood methodology. As stated in the analysis included in this report both gaps and lags are used. According to NCHRP 572, “The lags have been converted to gaps using an approximate follow-up headway” [13]. However, no mention is made of how the follow-up headway to convert the lags is found. The study from the University of Wisconsin’s TOPS lab makes mention of lags and clearly uses them in the analysis, but does not discuss adding time to the lags to make them in to approximate gaps [22]. Therefore, this analysis does not add any additional time to the lags.

In order to implement the methodology, the statistical software package “R” version 2.4.11, was used to perform a logistic regression on the accepted and rejected gaps [29]. For the logistic regression the data must be in the form of ones and zeros, with one (accepted gap/lag) defined as success and zero (rejected gap/lag) defined as not success. Therefore, all of the accepted and rejected gap and lag data was merged into one column in a csv file in Excel. Then a column was created to denote whether the gap was a success or not. If the gap or lag was accepted then the value in the success column for that cell was entered as one and if it was not accepted then the value was entered as zero. Figure 29 shows an example of the gap data plotted at ones and zeros.

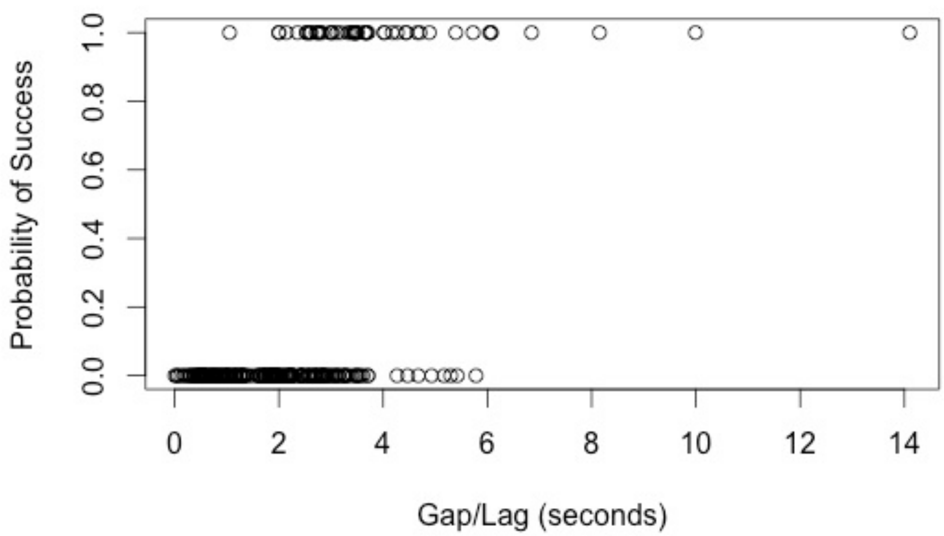

Figure 29. Data plotted as ones and zeros in R

The maximum likelihood for the critical gap is found at the inflection point on the logistic curve or the point where the curve changes from increasing at an increasing rate to increasing at a decreasing rate. This inflection point is found by finding where the second derivative of the logistic equation is equal to zero. R is used to find the equation for the logistic curve. The equation is in the form of the equation shown as Equation 12 [30]. The critical headway for the data is found at the inflection point on this curve. Figure 30 an example of a logistic curve with the inflection point marked.

$$E(Y_i|X_i) = \pi_i = \frac{e^{\beta_0+\beta_1 X_i}}{1+e^{\beta_0+\beta_1 X_i}} \qquad (12)$$

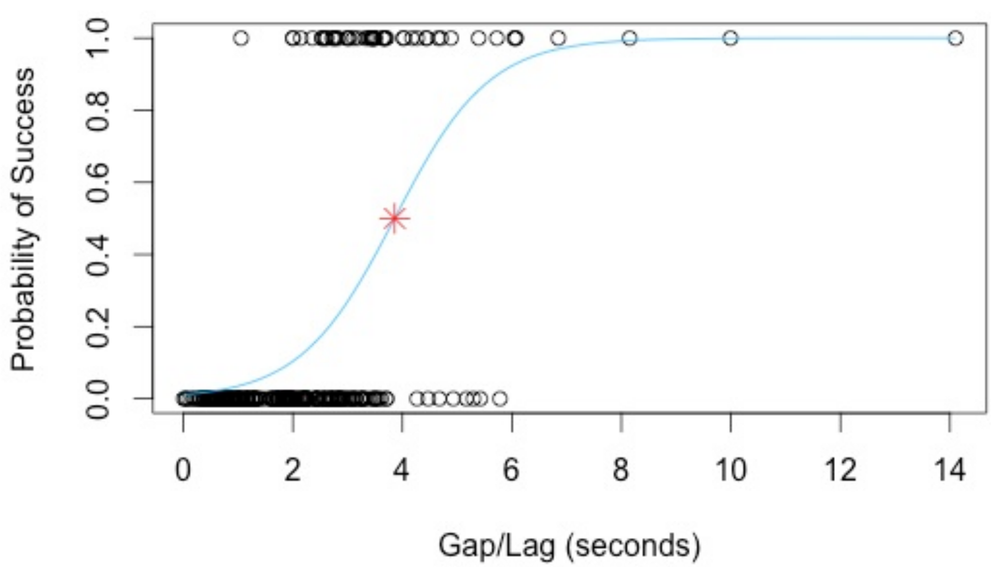

Figure 30. Logistic regression with inflection point

As can be seen from Figure 30, the length of the gap or lag is shown on the x-axis, and the probability of success i.e. the gap is accepted, is shown on the y-axis. Once the value of the critical headway at the inflection point is found, this value for critical headway can be plugged back into the original equation to find the y coordinate of the inflection point, which is the probability of success or the probability that a vehicle will accept that gap.

3.5 Follow-up Headway

Since the follow-up headway can be found directly from the timestamps collected in the field, the process selected for determining the follow-up headway was fairly straightforward. After the lag and gap information was determined from the timestamps, the queuing timestamps were used to determine which instances of follow-up headway should be used for data analysis, i.e. occurred under queued conditions. The keystroke “x” was used to mark the beginning of the queue and the keystroke “z” was used to denote the end of a queue. A queue for the purposes of determining follow-up headway for this project is defined as any vehicle that experiences delay at the roundabout as a result of the vehicle in front of it. After the queuing timestamps were collected, the csv file containing this information was opened. Then, the rows containing the “x” or beginning of queue timestamp were colored green and the rows containing the “z” or end of queue timestamp were colored red. Then the queuing timestamps were copied into the excel file with the other timestamps from the same video. The colors of the cells help to visually distinguish where the queuing occurs in the file, and only instances of follow-up headway that occur during the periods of queuing are used for this project. All other instances of follow up headway are removed from the file.

3.7 Model Calibration

After the critical headway and the follow-up headway are determined from the data, a weighted average from all of the roundabout approaches included in the analysis was found. For follow-up headway, the weighted average was calculated by multiplying the average follow-up headway found for each approach by the total number of data points on the approach. Then the results for all of the approaches were added and divided by the sum of the total number of data points at all approaches. The weighted average for the critical headway was found in a similar manner and the results of both weighted averages were plugged into the equations given in the HCM 2010 for the parameters of the single-lane roundabout capacity equation. After the parameters are calculated, they can be substituted into the equation to create a calibrated roundabout capacity equation for Georgia. The resulting equation requires only the circulating flow to determine the capacity on an approach.

CHAPTER 4

RESULTS & DISCUSSION

As part of the roundabout screening process, seven roundabouts were identified as the most likely to have sufficient traffic volumes based on their locations and AADTs found from the GDOT STARS website. Table 5, shows the five roundabouts identified along with their locations, cross streets and AADT.

Table 5. Possible Candidates for data collection

County	**City**	**GDOT District**	**Intersection**	**AADT**
Carroll	Carrollton	6	Newnan Rd./ Olympic Dr./Independence Dr.	Not available
Cobb	Acworth-Kennesaw	7	W. Sandtown Rd./Villa Rica Rd.	Not available
Douglas	Douglasville	7	SR 5/SR 166	5350
Gwinnett	Duluth	1	McClure Bridge Rd/ W Lawrenceville St/ Irvindale Rd NW	Not available
Fayette	Fayetteville	3	Grady Ave/Beauregard Ave.	8410
Fulton	Roswell	7	Grimes Bridge Rd/Norcross St./Warsaw Rd.	7860
Newton	Covington	2	Turner Lake Rd./Clark St.	8450

After field site visits, it was found that the roundabout in Carrollton is dominated by the through movements on the major street and with few conflicting movements at this roundabout. Therefore, data were not collected at this roundabout. Data were collected at the other six roundabouts shown in Table 5.

At each of these roundabouts, data were collected for approximately two hours on two different approaches. Data were collected during the PM peak period for all of the roundabouts except for the roundabout in Duluth. The roundabout in Duluth only has three legs. Based on knowledge of traffic patterns in the area, the research team expected that there would be more conflicting traffic in the morning than in the afternoon. Therefore, data were collected during the AM peak period for this roundabout. Table 6 shows the data collection time periods and dates along with which approaches were recorded.

Table 6. Roundabout data locations, approaches, and times

Location	Intersection	No. Legs	Date	Approach	Time
Fayetteville	**Grady Ave/ Beauregard Ave**	**4**	12/15/2011	Eastbound, Southbound	3:20PM – 5:45PM
			04/11/2012	Eastbound, **Southbound**	**4:30PM – 6:50PM**
Covington	**Turner Lake/ Clark Street**	**4**	**03/01/2012**	**Northbound, Southbound**	**4:20PM – 7:00PM**
			05/24/2012	Eastbound, Westbound	4:30PM – 6:45PM
Acworth-Kennesaw	Villa Rica/ Sandtown	4	03/27/2012	Southbound	4:20PM – 6:45PM
Douglasville	SR 166/ SR5	4	05/14/2012	Eastbound, Westbound	4:30PM – 7:00PM
Roswell	**Grimes Bridge/ Norcross/ Warsaw**	**5**	**05/15/2012**	**Southbound, Eastbound**	**4:30PM – 6:30PM**
Duluth	McClure Bridge Rd/ W Lawrenceville St/ Irvindale Rd NW	3	06/01/2012	Eastbound, Southbound	7:00AM – 9:00AM

At most of the roundabouts, there was little queuing observed on the approaches. Since the roundabout in Duluth has only three approaches there was little conflicting traffic observed during the AM peak period, although some queuing was observed. The highest rate of queuing was observed at the roundabouts in Roswell, Covington, and Fayetteville. Therefore, this analysis focuses on two approaches from both the Covington and Roswell roundabouts and one approach from the roundabout in Fayetteville. The approaches included in this study are shown in bold in Table 6. The eastbound approach from Fayetteville is not included in this study because this approach has a right turn bypass lane and a significant percentage of the vehicles that use this approach were observed to use this lane.

Two types of analysis were done on the studied approaches at these three roundabouts. One analysis that excluded exiting vehicles and the other analysis included exiting vehicles. Since adjustment is made to gaps that end with an exiting vehicle, Table 7 shows the distance the amount of time added to each affected gap at each roundabout approach. Covington has a larger inscribed diameter than the other two roundabouts and is in a higher speed location. Therefore, when calculating the added time, a speed of 20mph was used. A speed of 15mph was used for the approaches on both of the other roundabouts. Google Earth™ was used to measure the approximate distance between the point identified as the exiting point on aerial imagery and the confliction point.

Table 7. Correction Distances

Roundabout Approach	Correction Distance	Speed	Added Time
Fayetteville	50’	15	2.273
Covington SB	50’	20	1.705
Covington NB	55’	20	1.875
Roswell EB	50’	15	2.273
Roswell SB	55’	15	2.500

4.1 Covington

The first roundabout analyzed for this study is the roundabout at the intersection of Turner Lake Road and Clark Street in Covington, Georgia. This roundabout was opened in November 2010 [31]. A map showing the area surrounding this roundabout is shown in Figure 31. This roundabout is located near I-20 and there is a heavy volume of traffic using this roundabout in the PM peak period. An aerial image of this roundabout is shown in Figure 32. Data were collected on all approaches at this roundabout. However, since this roundabout is so close to I-20, the queuing at this roundabout during the peak PM period was predominately observed on the southbound and northbound approach. Therefore, the data analysis for this study includes data from the northbound and southbound approaches.

Figure 31. Map of Covington roundabout location (source: Google Earth™, accessed 6/28/2012)

Figure 32. Aerial image of Covington roundabout (Source: Google Earth™, accessed 6/28/2012)

4.1.1 Data Collection

The data collection for the roundabout in Covington occurred on March 1, 2012. Figure 33 shows the placement of the cameras around the perimeter of the roundabout. The camera denoted by the "1" was set on the southwest corner of the roundabout and used to record the operations on the southbound leg. The other camera, denoted by the "2" was placed on the northeast corner of the roundabout and used to record operations on the northbound leg. Data were collected between 4:20PM and 7:00PM on both legs. The cameras were in place for more than two hours because there was still significant traffic at the roundabout at 7:00PM. However, data collection ended at approximately 7PM due to loss of daylight.

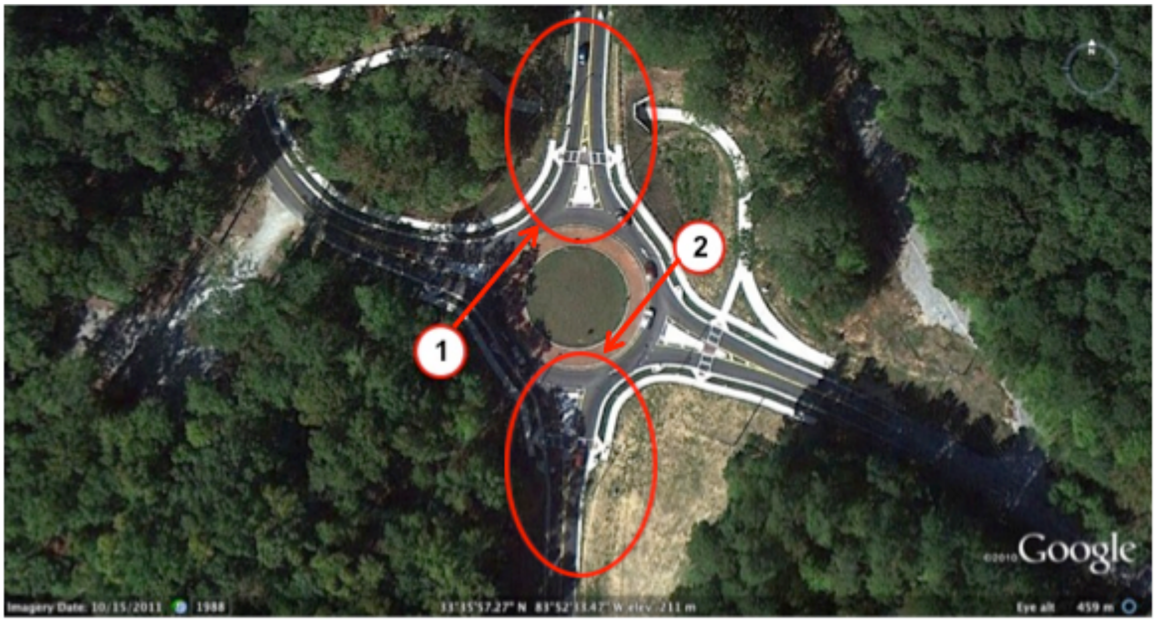

Figure 33. Camera locations for Covington (Source: Google Earth™)

4.1.1 Follow-up Headway

Table 8 shows a summary of the follow-up headway extracted from the videos recorded at the Covington roundabout. Table 1 includes the average follow-up headway found for the entire video length and the average follow-up headway found only during periods of queuing. Table 8 also shows the average follow-up headway for the cases when exiting vehicles were excluded and for when they were included in the analysis. Additionally, the table shows the number of data points gathered in each case as well as the total amount of time that the videos were queued.

Data were collected on these two approaches for over two hours. On the Southbound approach data were collected for 158 minutes. The total length of queuing observed on the southbound approach was 114.46 minutes, which means that this approach had observable queuing for approximately 72.4% of the data collection period.

On the north bound approach the total length of the video data collected is 144.82 minutes. The northbound approach was queued for 93.43 minutes, so this approach was queued for 64.5% of the data collection period. The length of the queuing both in terms of time and number of cars was greater than the queuing observed at the other roundabouts.

Table 8. Follow-up headway for Covington roundabout

	All observations		Observations during periods of queuing		Total Minutes of Queuing
	Average (sec)	n	Average (sec)	n	
Excluding Exiting Vehicles					
Southbound	3.72	1550	3.40	1332	114.46
Northbound	4.49	1229	3.75	983	93.43
Considering Exiting Vehicles					
Southbound	3.00	677	2.81	600	114.46
Northbound	2.95	320	2.80	270	93.43

Given the proportion of time that this roundabout is queued in the peak period, there is an abundance of follow-up headway data. However, since the through movement on Turner Lake Road is the dominant traffic movement at this roundabout, there is also a high proportion of exiting vehicles. When exiting vehicles are included the number of observations of follow-up headway during the queuing periods drops from 1332 observations to 600 observations on the southbound approach and from 983 to 270 on the northbound approach. Recall, follow-up headway is a measurement of the headway of two vehicles in a queue entering the roundabout consecutively with no other events between them. Thus, when exiting vehicles are included in the analysis, if a vehicle exits the roundabout between two cars entering the roundabout, then the follow-up headway is not collected for the two entering vehicles. Thus, when exiting vehicles are included there is a noticeable drop in the amount of data collected for follow-up headway.

Figure 34 and Figure 35 show density plots of the follow-up headway on the northbound approach of the Covington roundabout. When queuing is accounted for, the majority of follow-up headway observations fall between two and four seconds. However, when the follow-up headway observations that do not occur during periods of queuing are included, there is a significant portion of follow-up headway observations in excess of four seconds. Therefore, even though this roundabout was queued for a significant portion of the data collection period, it is still necessary to remove observations of follow-up headway that did not occur during periods of queuing to get an accurate measurement of follow-up headway for this approach.

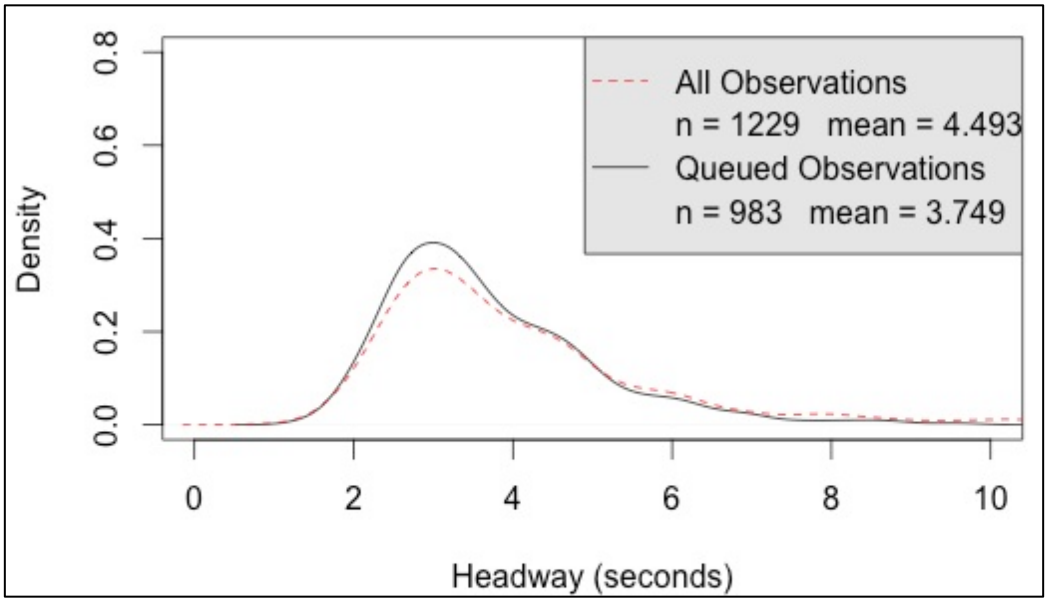

Figure 34. Covington NB density plot for follow-up headway excluding exiting vehicles

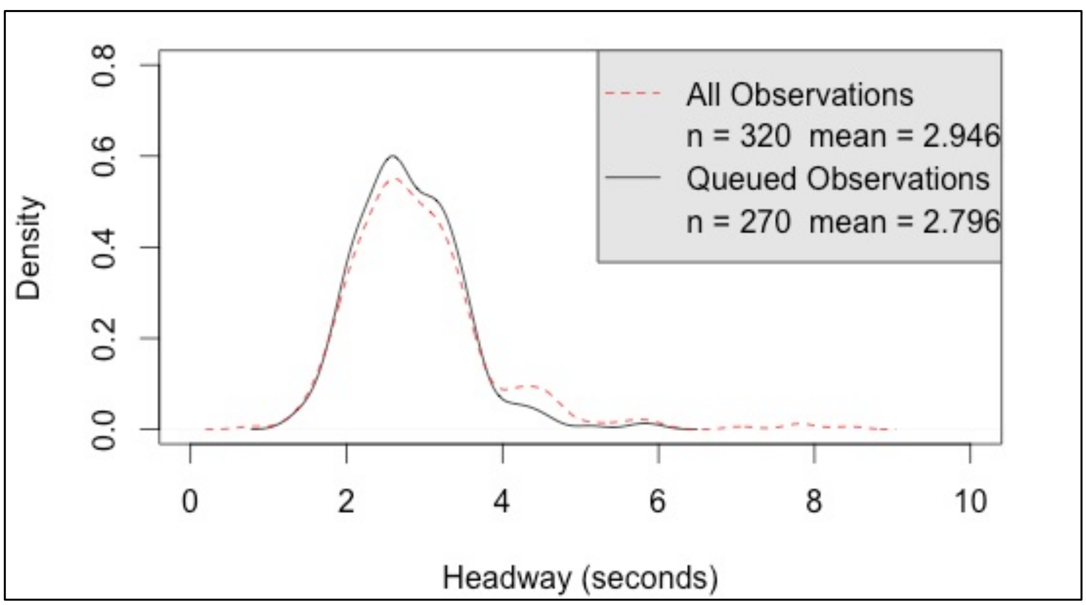

Figure 35. Covington NB density plot follow-up headway including exiting vehicles

Similar trends are seen for the density plots for the southbound approach. Figure 36 shows the density plots for the case where exiting vehicles are excluded on the southbound approach. Figure 37 shows the density plot for the southbound approach for the case in which exiting vehicles are included.

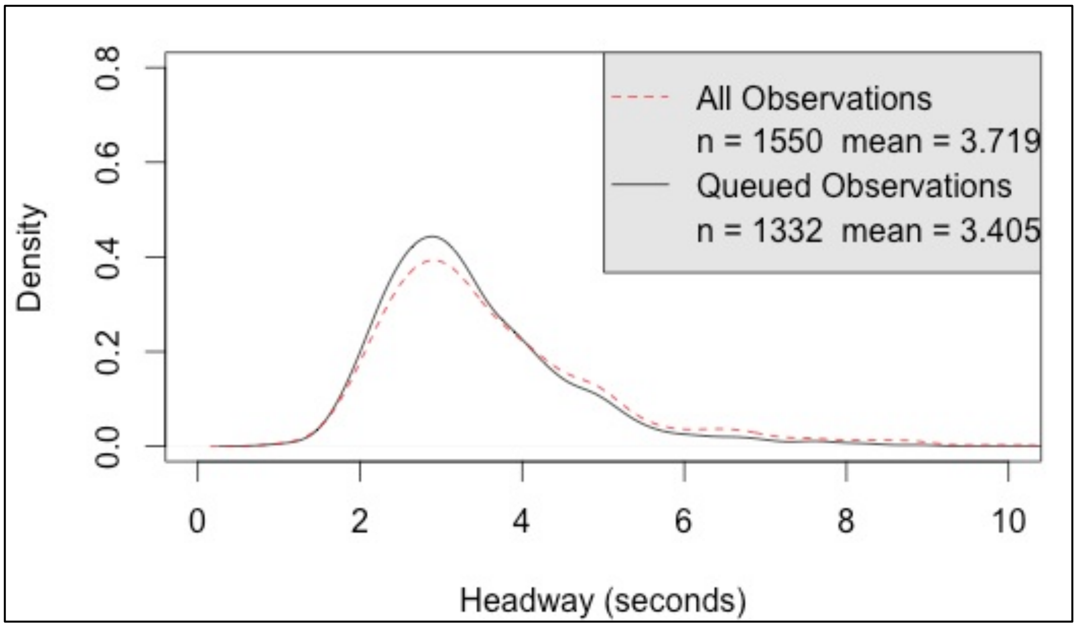

Figure 36. Covington Southbound density plot for follow-up headway excluding exiting vehicles

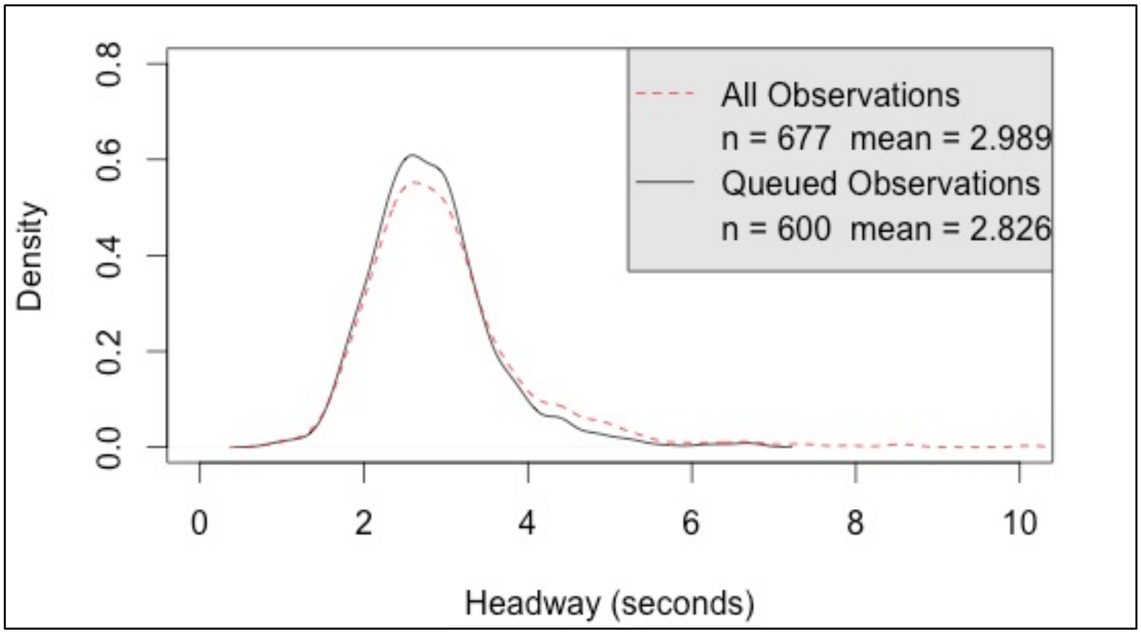

Figure 37. Covington Southbound density plot for follow-up headway including exiting vehicles

When the exiting vehicles are excluded in the analysis the follow-up headway found for the southbound approach is 3.4 seconds and the follow-up headway for the northbound approach is found to be 3.75 seconds. This is a difference of 0.35 seconds. However, when exiting vehicles are included the follow-up headway drops to 2.8 seconds for the southbound approach and 2.81 seconds for the northbound approach. When exiting vehicles are included the follow-up headway for both approaches is the same. The observed drop in the average follow-up headway is a 0.59 second drop for the southbound approach and a 0.95 second drop for the southbound approach. Thus, exiting vehicles are observed to affect the follow-up headway on both approaches at this roundabout.

4.1.2 Critical Headway

Table 9 shows the gap data that was extracted for the roundabout in Covington. Since gap data can be collected regardless of whether or not the roundabout approach is queued, there is no distinction made between data that was collected during a queued period as there is with the follow-up headway. As with the follow-up headway the case in which exiting vehicles are included and for which exiting vehicles are excluded is presented for each approach.

Table 9. Gap/lag data for Covington

	Southbound				Northbound			
	Excluding Exiting		Including Exiting		Excluding Exiting		Including Exiting	
	Avg (sec)	n	Avg (sec)	n	Avg (sec)	n	Avg (sec)	n
Accepted Gaps	6.32	10	4.52	259	7.74	20	5.04	343
Accepted Lags	3.34	104	3.27	644	4.98	94	3.25	600
Rejected Gaps	2.68	75	2.93	277	2.82	45	3.05	328
Rejected Lags	1.80	243	2.15	577	2.34	198	2.65	604

A logistic regression was performed on the gap data. Two regressions were performed, one on the set of data where exiting vehicles were excluded and the other where exiting vehicles were included. Figure 38 and Figure 39 show the logistic regressions for the northbound approach of the Covington roundabout.

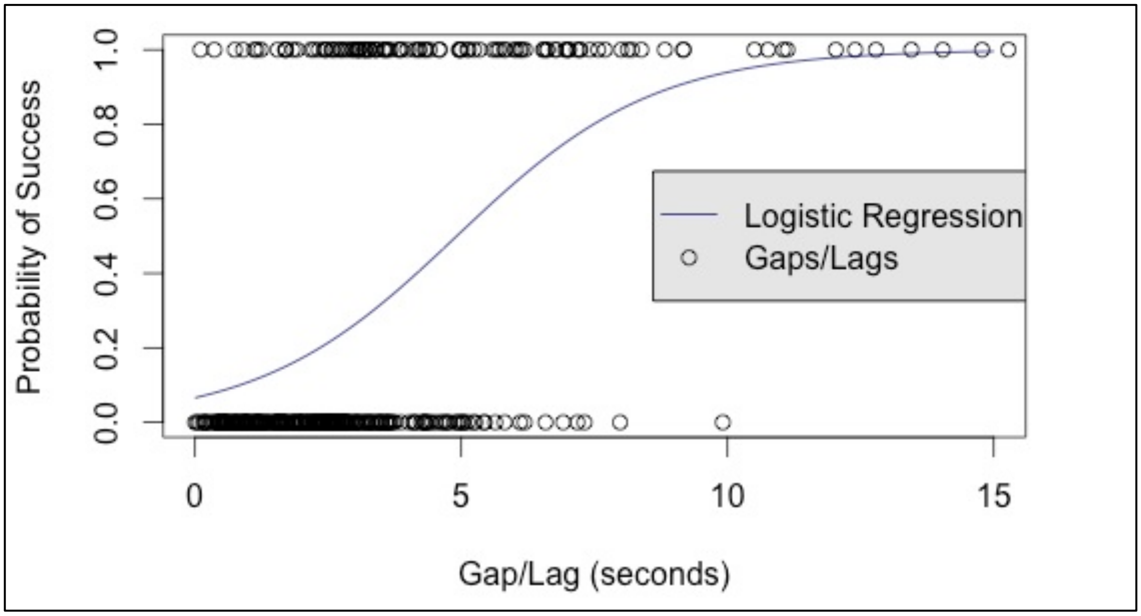

Figure 38. Covington NB logistic regression excluding exiting vehicles

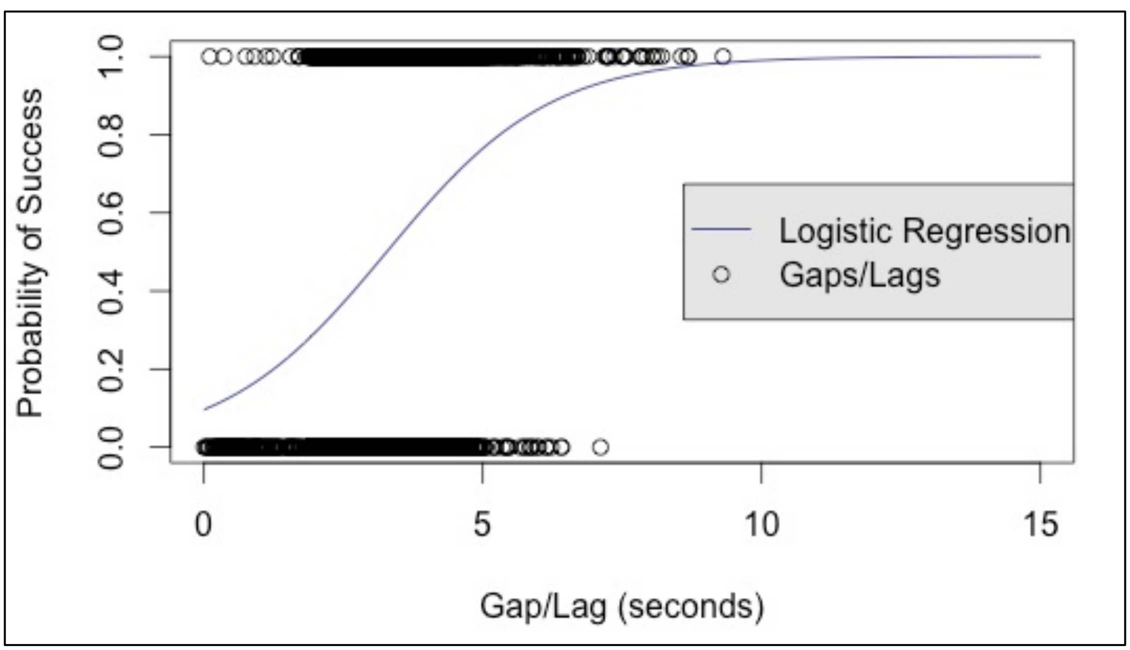

Figure 39. Covington NB logistic regression including exiting vehicles

Table 10 shows the parameters for the logistic curves shown in Figure 38 and in Figure 39. The critical headway that is found by taking the second derivative of each equation is also shown in the table. When the exiting vehicles are excluded in the analysis the critical headway is found to be 4.91 seconds. By contrast, when exiting vehicles are included in the analysis the critical headway is found to be 3.28 seconds. This constitutes a difference of 1.63 seconds, which indicates that for the northbound approach of the Covington roundabout exiting vehicles have an effect on the critical headway as well as the follow-up headway.

Table 10. Parameters for Covington NB logistic regressions

Coefficients	Estimate	Std. Error	z value	Pr(>|z|)	Critical Headway (sec)
Covington NB excluding exiting					
Intercept	-2.65144	0.27754	-9.553	<10e-6***	4.91
x	0.53948	0.07027	7.677	<10e-6***	
Covington NB Including exiting					
Intercept	-2.24191	0.15383	-14.57	<10e-6***	3.28
x	0.68406	0.04474	15.29	<10e-6***	

The logistic curves for the southbound approach of the Covington roundabout are shown in Figure 40 and Figure 41. The curve for the case where exiting vehicles is included in the analysis is steeper than the curve for the case when exiting vehicles are excluded. This trend was also seen in the logistic curves for the northbound approach.

Table 11 shows the parameters for the logistic regressions for the southbound approach. The critical headway found for each case is also presented in the table. When exiting vehicles are excluded in the analysis, the critical headway is found to be 4.73

seconds and when exiting vehicles are included, the critical headway is found to be 2.9 seconds. The difference between the two cases is 1.83 seconds, which is 0.2 more seconds of difference between the two cases than was seen for the northbound approach. Additionally, for both cases, the critical headway is found to be lower for the southbound approach than for the northbound approach. One possible reason that the critical headway is found to be lower is that the southbound approach has the most traffic by volume and the most queuing during the PM peak period. Therefore, since there is more traffic on that approach, the vehicles may be willing to accept smaller gaps.

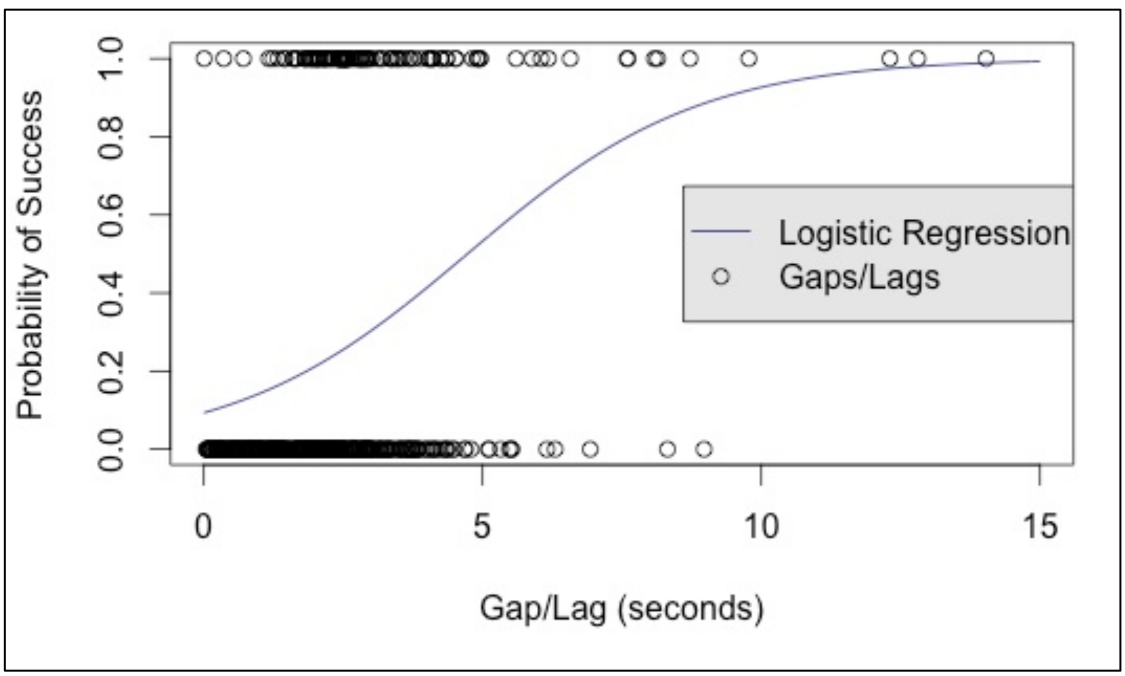

Figure 40. Covington SB logistic regression excluding exiting vehicles

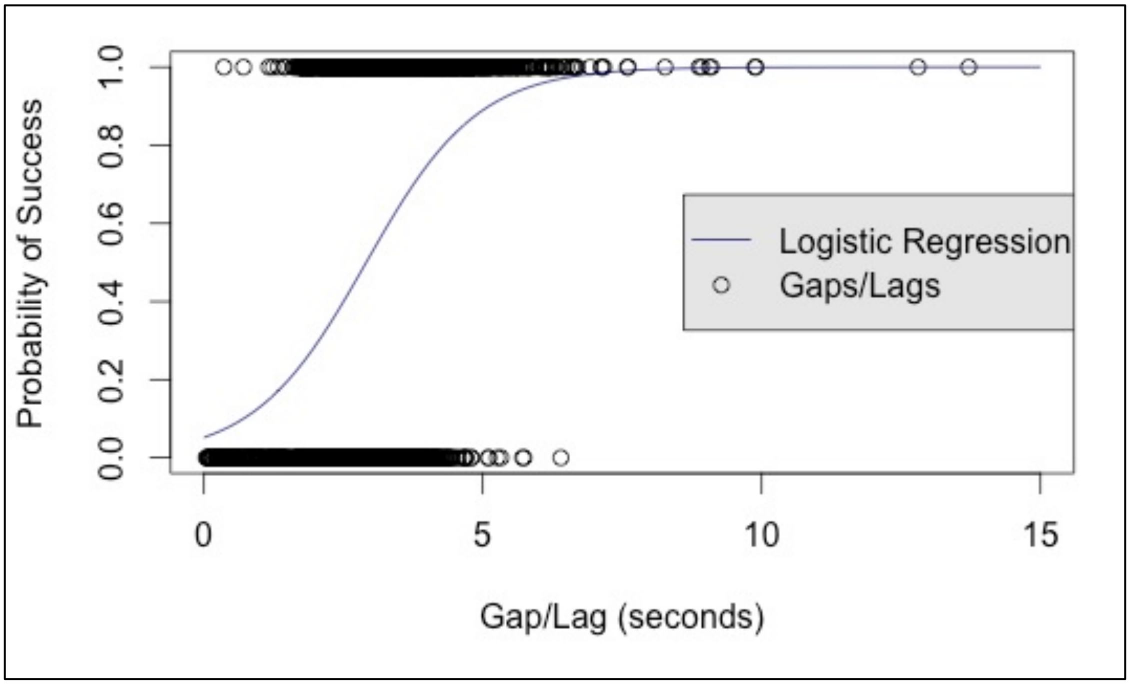

Figure 41. Covington SB logistic regression including exiting vehicles

Table 11. Parameters for Covington SB logistic regressions

Coefficients	Estimate	Std. Error	z value	Pr(>|z|)	Critical Headway (sec)
SB excluding exiting					
Intercept	-2.27419	0.23594	-9.639	<10e-6***	
x	0.4811	0.07739	6.217	<10e-6***	4.73
SB Including exiting					
Intercept	-2.90545	0.17569	-16.54	<10e-6***	
x	0.99875	0.05736	17.41	<10e-6***	2.9

4.2 Roswell

The roundabout in Roswell, Georgia is at the intersection of Grimes Bridge Road, Norcross Street, Warsaw Road, and Melody Lane. This roundabout is a five-leg roundabout that was opened in June 2011 [32]. The approach on Melody Lane has much less traffic than the other four legs because this approach is just a residential street that is not a through street. Figure 42 is a map showing the area surrounding the roundabout in Roswell. This roundabout is in a residential area and located adjacent to a church. This roundabout exhibits queuing on all of the four major legs during the PM peak. Figure 43 shows an aerial image of the Roswell roundabout.

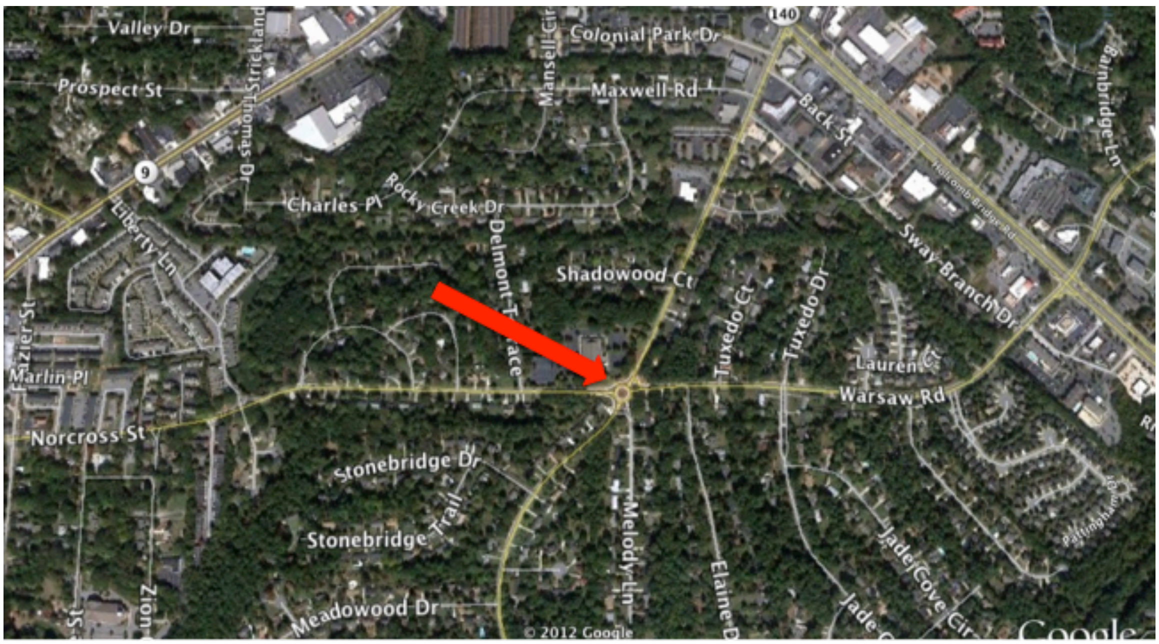

Figure 42. Map of location of Roswell roundabout (Source: Google Earth™, accessed 6/28/2012)

Figure 43. Aerial image of Roswell roundabout (Source: Google Earth™, accessed 6/28/2012)

4.2.1 Data Collection

Data collection for the Roswell roundabout occurred on May 15, 2012. Video data were recorded between 4:45PM and 6:00PM. The cameras were set up to record operations on the southbound and eastbound approaches. These legs were chosen because they were observed to have significant queuing as well as significant volumes of conflicting vehicles. Also, neither of these approaches has a slip lane. However, it was observed for the westbound approach that few cars turn right and the slip lane was most likely built to compensate for the severe angle between the approaches at this roundabout.

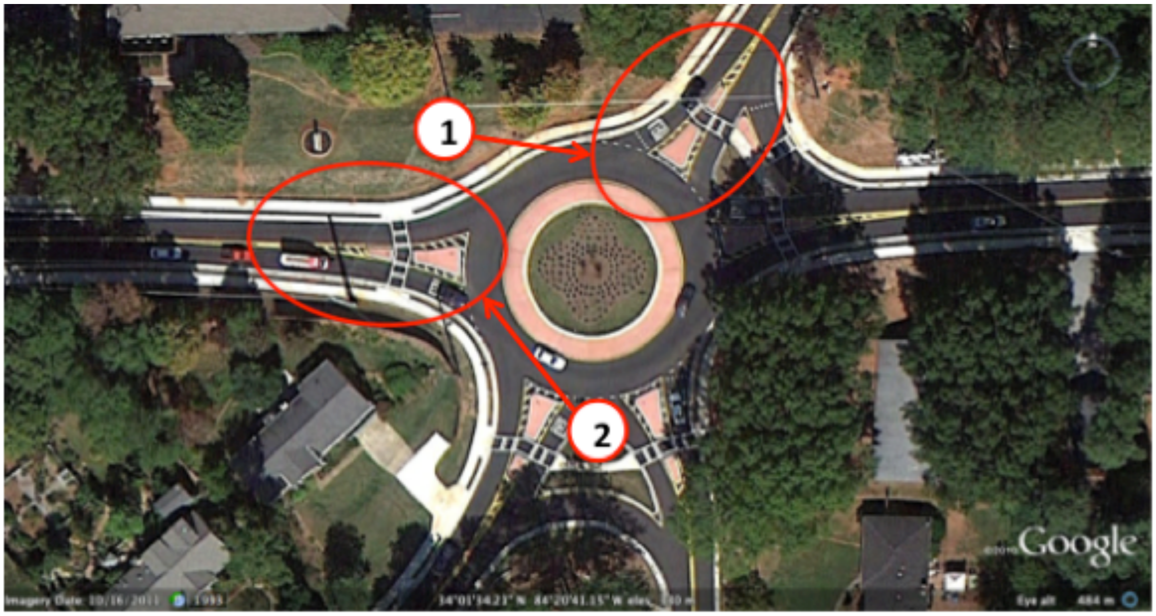

Figure 44. Camera locations for the Roswell roundabout (Source: Google Earth™, accessed 6/28/2012)

4.2.2 Follow-up Headway

Table 12 shows a summary of the average follow-up headway found for the Roswell roundabout for both the southbound and eastbound approaches. As with the Covington roundabout, average follow-up headway is shown for the case where exiting vehicles are excluded and the case where they are included both for the entire duration of the data collection and only during periods of queuing. The total length of the video for the southbound approach is 126.33 minutes. The total minutes of queuing on this approach is 40.17 minutes, which means that the approach was queued for 31.8% of the data collection period. The total length of the video on the southbound approach is 126.42 minutes and the total length of queuing on this approach is 33.17 minutes. The southbound approach was queued for 26.2% of the data collection period. There are much fewer data points for follow-up headway collected at the Roswell roundabout than the Covington roundabout. The reason that there is much less follow-up headway data at the Roswell roundabout is because the approaches at the Covington roundabout were queued for more than twice as much time as the Roswell roundabout. Also, the Roswell roundabout is less dominated by a major movement than the Covington roundabout, which allows for more data collected pertaining to gap acceptance but fewer data points for follow-up headway.

Similar trends are seen in the data for Roswell as was seen in the follow-up headway data for Covington. Removing all the observations of follow-up headway that do not occur during periods of queuing reduces the averages follow-up headway. Also, including exiting vehicles in the analysis also reduces the average follow-up headway. When both queuing and exiting vehicles are included in the analysis, there is only a difference of 0.14 seconds between the follow-up headway on the southbound and eastbound approaches.

Table 12. Summary of follow-up headway for Roswell roundabout

	All observations		Observations during periods of queuing		Total Minutes of Queuing
	Average (sec)	n	Average (sec)	n	
Excluding Exiting Vehicles					
Southbound	3.56	545	3.03	260	40.17
Eastbound	4.77	683	3.30	297	33.17
Including Exiting Vehicles					
Southbound	3.16	422	2.74	195	40.17
Eastbound	2.88	230	2.60	121	33.17

Figure 45 and Figure 46 show the density plots for follow-up headway for the southbound approach at the Roswell roundabout. Figure 47 and Figure 48 show the density plots for the eastbound approach.

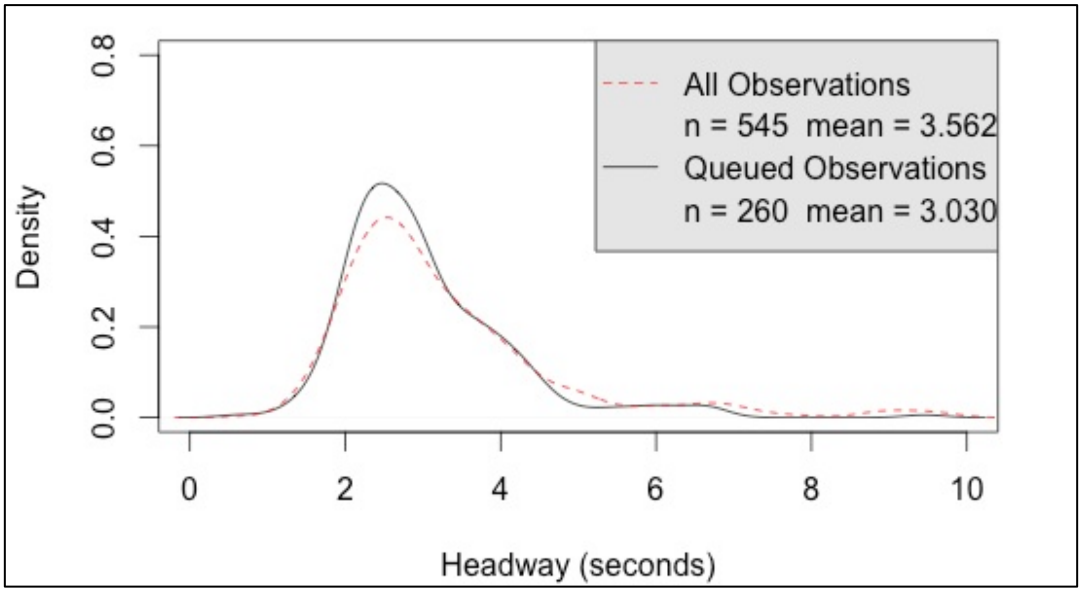

Figure 45. Roswell SB follow-up headway excluding exiting vehicles

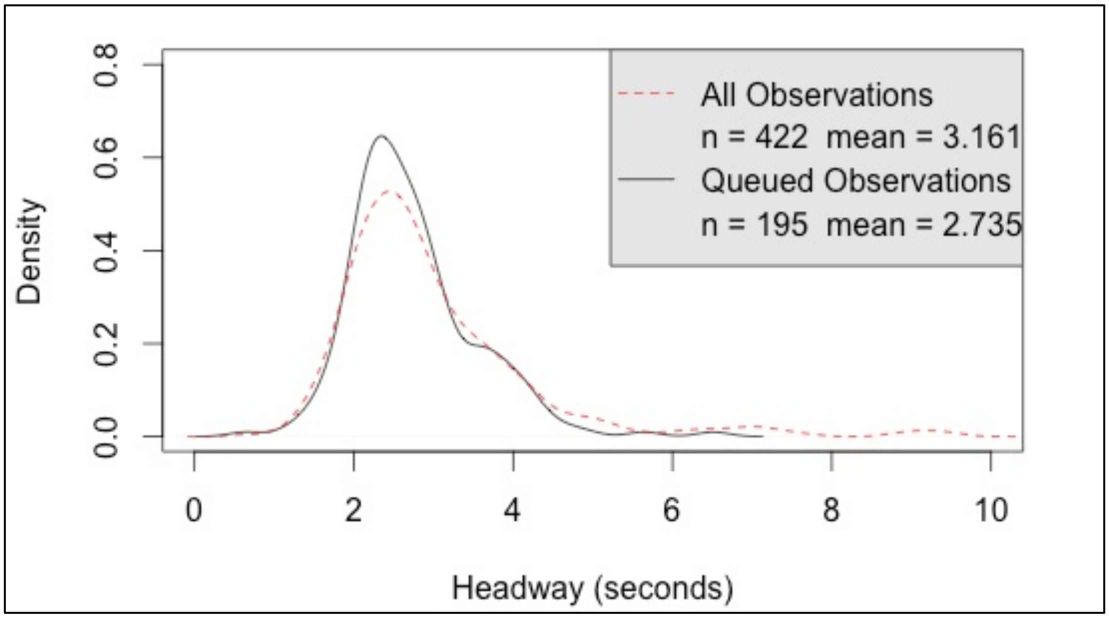

Figure 46. Roswell SB follow-up headway excluding exiting vehicles

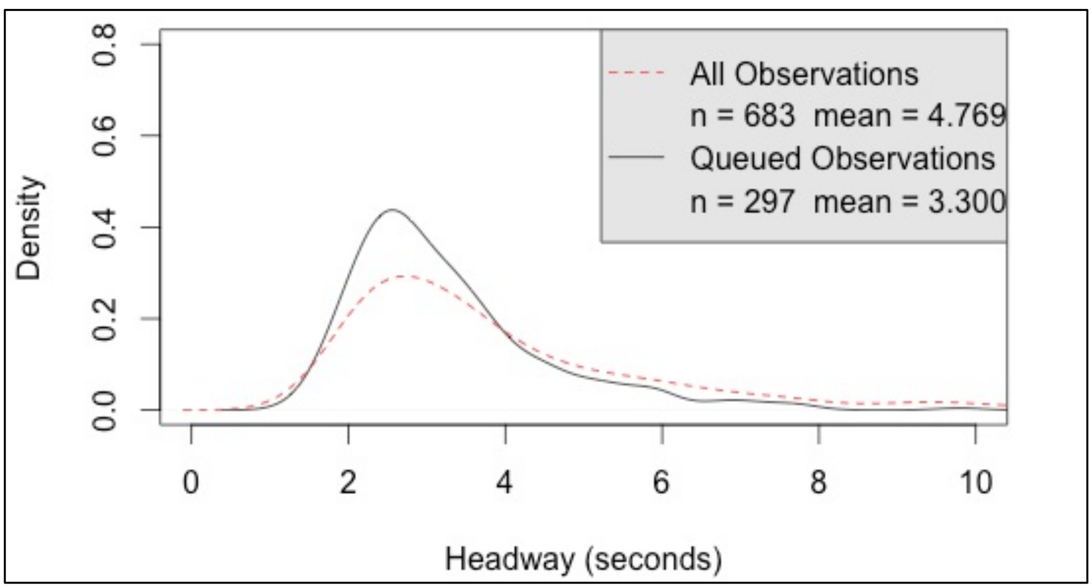

Figure 47. Roswell EB follow-up headway excluding exiting vehicles

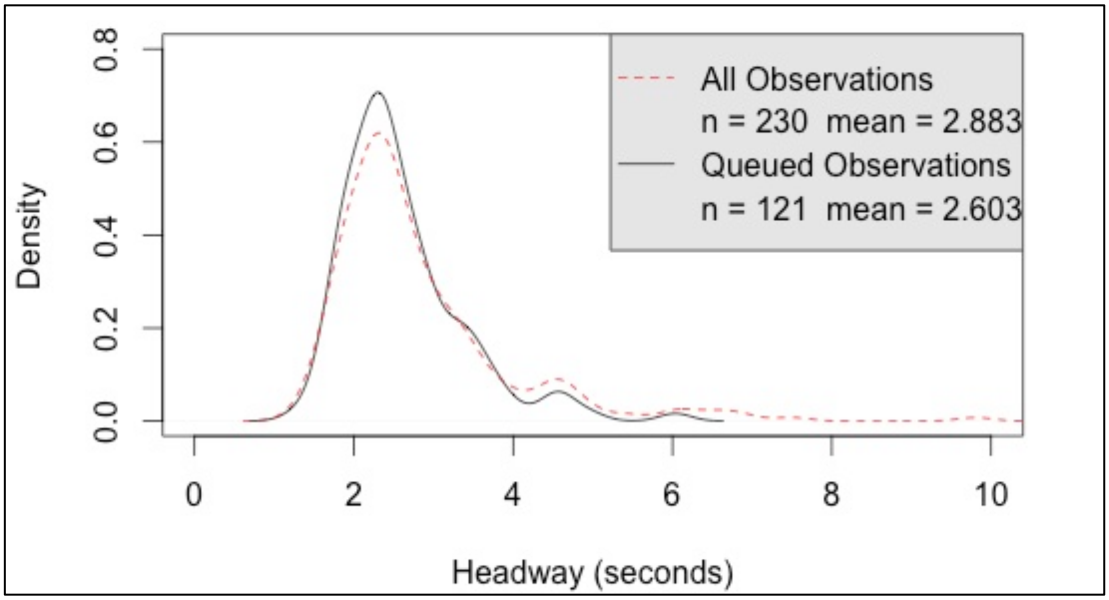

Figure 48. Roswell EB follow-up headway including exiting vehicles

4.2.3 Critical Headway

Table 13 shows the summary of gap data for the Roswell roundabout for both the southbound and eastbound approaches. The average gaps for the case when exiting vehicles are excluded and for the case when exiting vehicles are included are shown. As with the data from Covington, when exiting vehicles are included, the average accepted gap and the average accepted lag are shorter while the inclusion of exiting vehicles in the analysis tends to lengthen the average rejected gaps and the average rejected lags.

Table 13. Summary of gap/lag data for Roswell roundabout

	Southbound				Eastbound			
	Excluding Exiting		Including Exiting		Excluding Exiting		Including Exiting	
	Avg (sec)	n	Avg (sec)	n	Avg (sec)	n	Avg (sec)	n
Accepted Gaps	6.09	113	5.42	166	6.46	13	5.13	200
Accepted Lags	5.72	136	4.88	208	9.11	138	4.09	408
Rejected Gaps	2.62	168	2.88	229	2.75	25	3.44	207
Rejected Lags	1.64	309	1.93	360	1.80	125	2.53	313

Figure 49 and Figure 50 show the logistic regressions for the gap data on the eastbound approach of the Roswell roundabout. These graphs look similar, which indicates that exiting vehicles do not have as significant an impact at this roundabout as they do at the roundabout in Covington. Table 14 shows the logistic regression parameters along with the critical headway that was from these equations. The critical headway on the eastbound leg is 3.47 seconds when exiting vehicles are excluded in the analysis and 3.38 seconds when exiting vehicles are included in the analysis. This in only a difference of 0.11 seconds, which is much lower than the difference between the critical headway values observed between the two cases at the Covington roundabout. Therefore, exiting vehicles do not have a large impact on the eastbound leg of the Roswell roundabout.

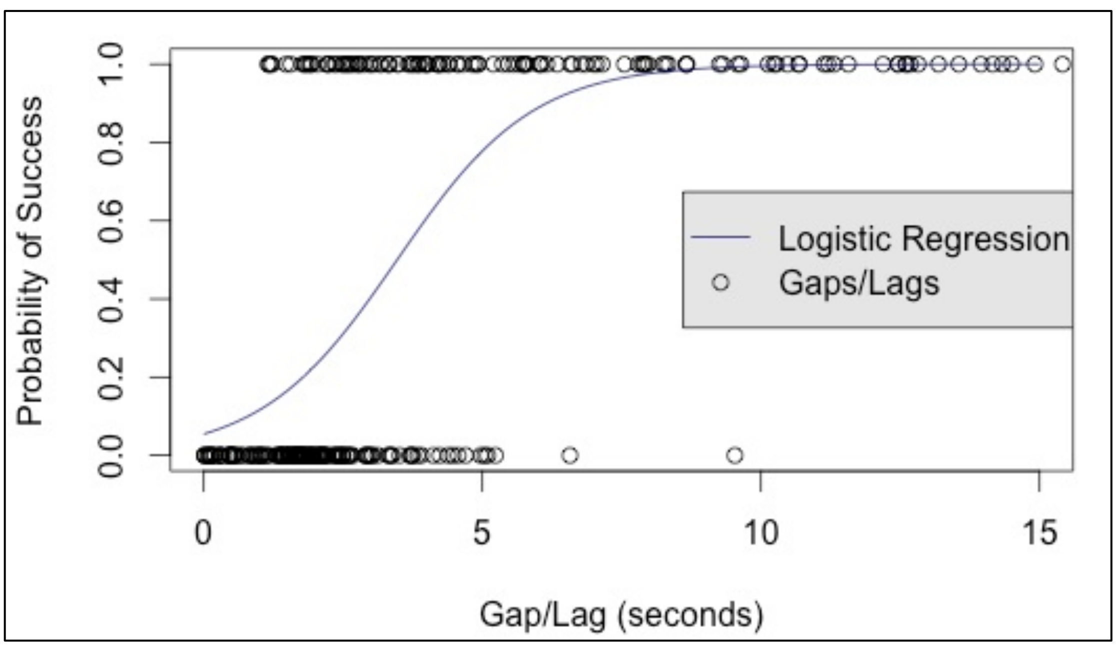

Figure 49. Roswell EB logistic regression excluding exiting vehicles

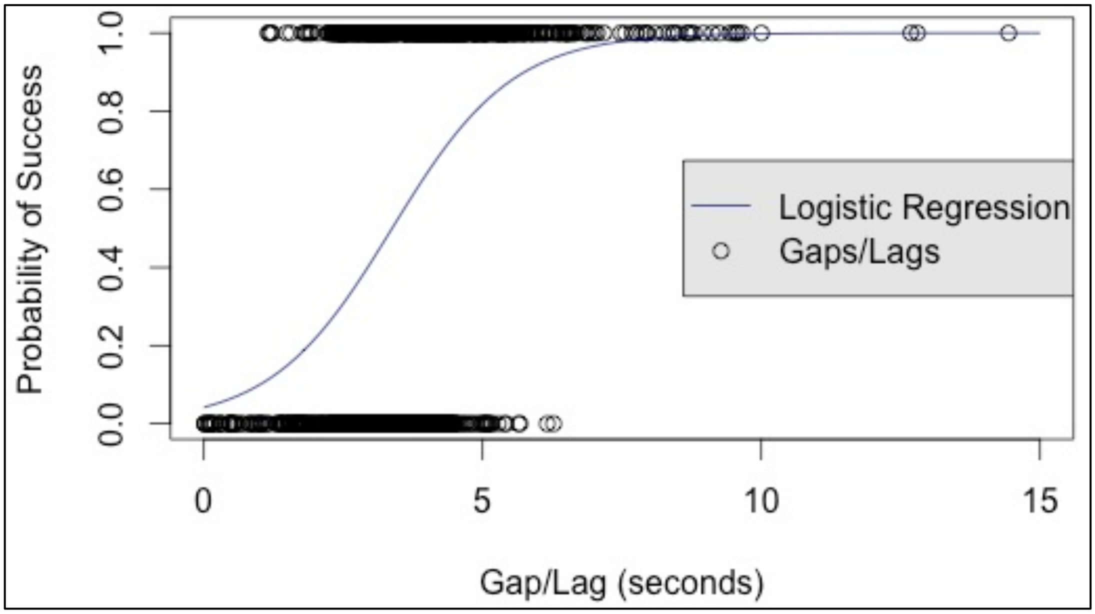

Figure 50. Roswell EB logistic regression including exiting vehicles

Table 14. Roswell EB logistic regression parameters and critical headway

Coefficients	Estimate	Std. Error	z value	Pr(>|z|)	Critical Headway (sec)
EB excluding exiting					
Intercept	-2.8496	0.3521	-8.093	<10e-6***	3.47
x	0.8211	0.1093	7.512	<10e-6***	
EB including exiting					
Intercept	-3.12799	0.2415	-12.95	<10e-6***	3.38
x	0.92649	0.06714	13.8	<10e-6***	

Figure 51 shows the logistic regression for the southbound approach of the Roswell roundabout when exiting vehicles are excluded and Figure 52 shows the logistic regression on the southbound leg when exiting vehicles are included in the analysis. These curves look very similar. Table 15 contains the parameters for these two logistic regression lines as well as the critical headway from the equations. The parameters for both equations are almost identical and the critical headway found for the case ignoring exiting vehicles is 3.75 seconds and the critical headway found for the case including exiting vehicles is 3.82 seconds. It is unexpected to find longer critical headway when exiting vehicles are included. This finding indicates that the method for including exiting vehicles in the analysis should be revisited.

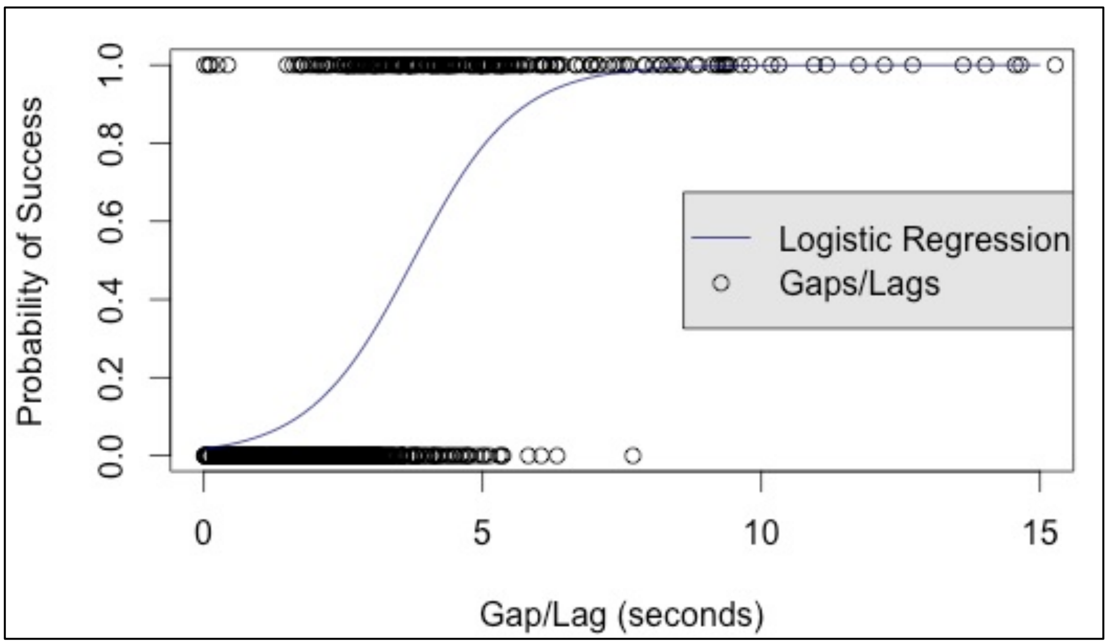

Figure 51. Roswell SB logistic regression excluding exiting vehicles

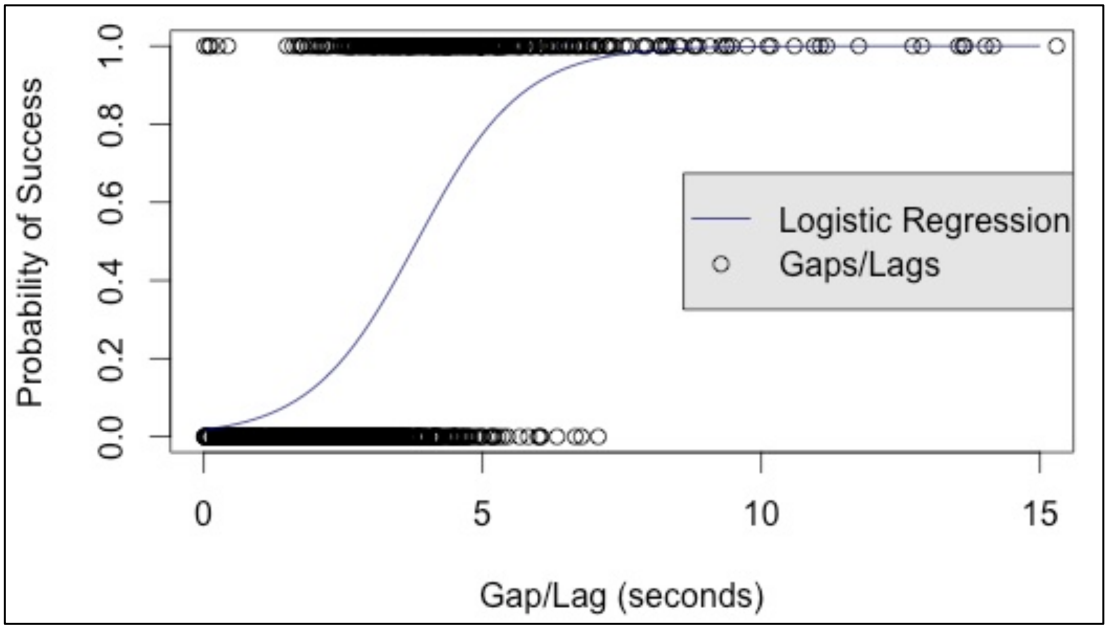

Figure 52. Roswell SB logistic regression including exiting vehicles

Table 15. Roswell SB logistic regression parameters and critical headway

Coefficients	Estimate	Std. Error	z value	Pr(>|z|)	Critical Headway (sec)
SB excluding exiting					
Intercept	-4.02094	0.28547	-14.09	<10e-6***	
x	1.07115	0.08511	12.59	<10e-6***	3.75
SB including exiting					
Intercept	-4.01732	0.25913	-15.5	<10e-6***	
x	1.05121	0.07153	14.7	<10e-6***	3.82

4.3 Fayetteville

The roundabout in Fayetteville opened in August of 2011 [33]. It is located at the intersection of Grady Avenue and Beauregard Avenue. This roundabout has four legs. The traffic at this roundabout, especially on the southbound approach, was observed to arrive in platoons presumably because of the close proximity of this roundabout to signalized intersections. Figure 54 shows an aerial image of the roundabout in Fayetteville. As can be seen from this image, there is a right turn bypass lane on the eastbound approach.

Figure 53. Map of the Fayetteville roundabout location (Source: Google Earth™, accessed 6/28/2012)

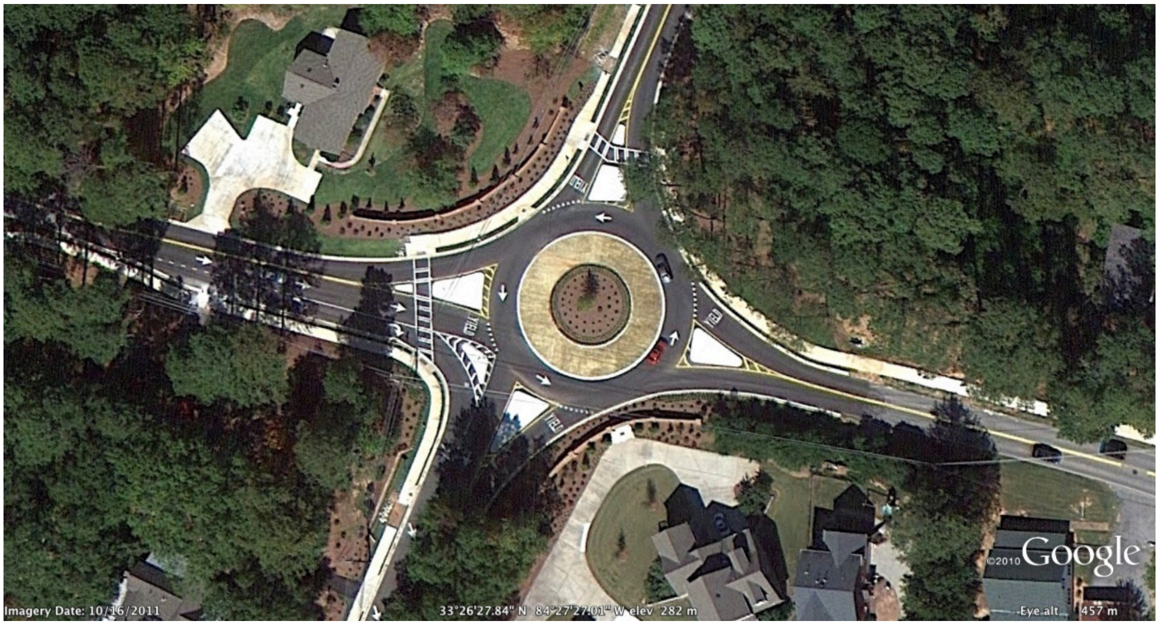

Figure 54. Aerial image of Fayetteville roundabout (Source: Google Earth™, accessed 6/28/2012)

4.3.1 Data Collection

Data collection for the roundabout in Fayetteville occurred on December 15, 2011 and again on April 11, 2012. This analysis considers only data on the southbound approach for the second data collection period. The first data collection period occurred during the winter when the days were shorter and therefore data collection had to begin early and end at approximately 5:30PM, when it became too dark to collect usable videos. However, there was significant volume observed after 5:30PM. Therefore, the data collection team returned to the roundabout when the daylight was longer to collect additional data. Also, basis on the previous data collection period, the camera angles were changed slightly to better capture the operations on each approach. Figure 55 shows the location of the camera capturing operations on the southbound approach at the Fayetteville roundabout. This location was ideal for data collection because it was several feet higher than the roundabout. Also, there were many trees and other foliage that disguised the camera and tripod from passing motorists.

Figure 55. Camera location for data collection at the Fayetteville roundabout (Source: Google Earth™, accessed 6/28/2012)

4.3.2 Follow-up Headway

Table 16 shows a summary of the follow-up headway for the southbound approach of the Fayetteville roundabout. This approach was queued for 20.35 minutes out of a total of 124.62 minutes of video. Thus queuing accounted for approximately 16.3% of the data collection period, which is the least amount of queuing observed at the three roundabouts. The fact that this roundabout did not have a standing queue for most of the data collection period is reflected in that the average follow-up headway drops by 2.25 seconds when queuing is accounted for when exiting vehicles are excluded and by two seconds when exiting vehicles are included.

Exiting vehicles are observed to only have a slight effect on the follow-up headway at this roundabout. During the periods of queuing, the follow-up headway only decreases by 0.14 seconds when exiting vehicles are included. Therefore, the effect of exiting vehicles at this roundabout is not as noticeable as it is at the other two roundabouts that are considered.

Table 16. Follow-up headway data for Fayetteville roundabout

	All observations		Observations during periods of queuing		Total Minutes of Queuing
	Average (sec)	n	Average (sec)	n	
Excluding Exiting Vehicles					
Southbound	5.37	369	3.12	176	20.35
Including Exiting Vehicles					
Southbound	4.98	318	2.98	153	20.35

Figure 56 and Figure 57 show the density plots for the follow-up headway at the Fayetteville roundabout. Figure 56 shows the follow-up headway when exiting vehicles are excluded and Figure 57 shows the follow-up headway when exiting vehicles are included in the analysis. These two plots are very similar in shape, which further indicates that exiting vehicles do not have a huge impact on the operations of the southbound leg of the Fayetteville roundabout.

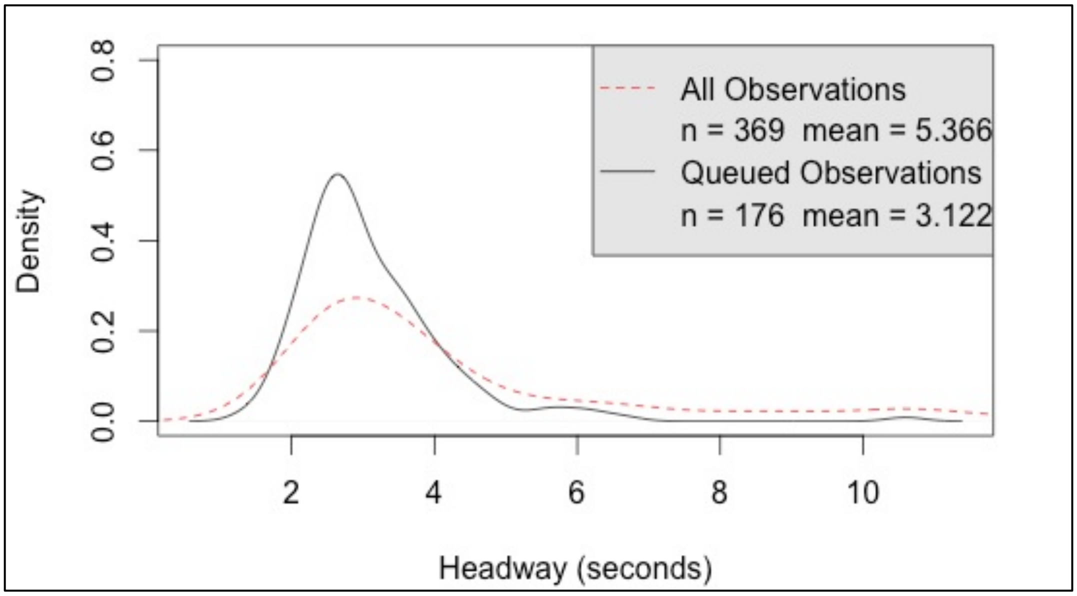

Figure 56. Fayetteville SB follow-up headway excluding exiting vehicles

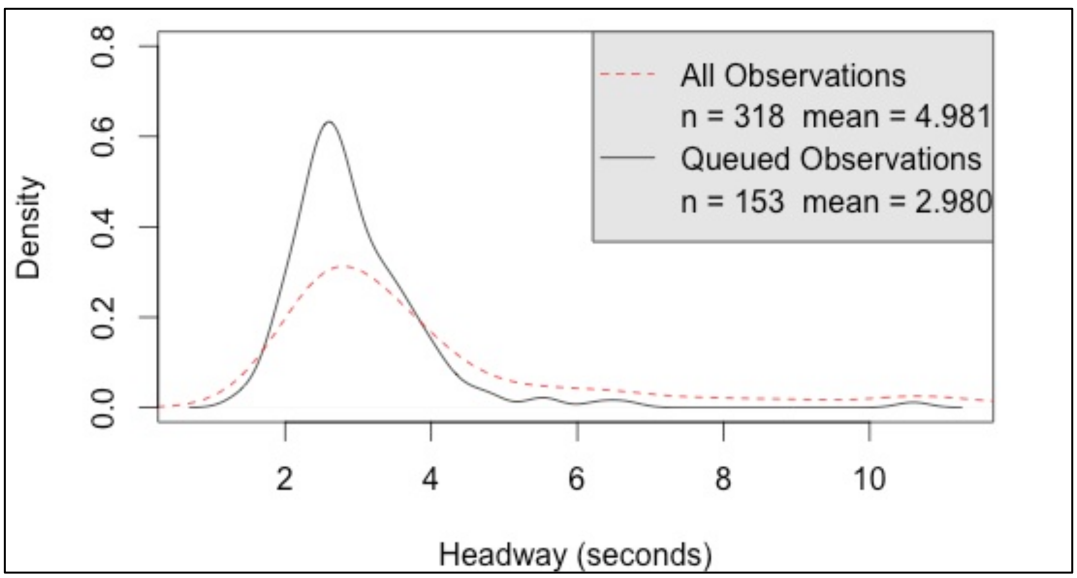

Figure 57. Fayetteville SB follow-up headway including exiting vehicles

4.3.3 Critical Headway

Table 17 is a summary of the gap data for the southbound leg of the Fayetteville roundabout. The average accepted gaps and average accepted lags are much longer than those observed at other roundabouts. This trend is likely due to the reduced prevalence of queuing seen at this roundabout as compared to other roundabouts. Including exiting vehicles in the analysis decreased the length of the accepted gaps and the accepted lags but the rejected gaps and the rejected lags are about the same between the two cases.

Table 17. Summary of gap/lag data for Fayetteville roundabout

	Southbound			
	Excluding Exiting		Including Exiting	
	Avg (sec)	n	Avg (sec)	n
Accepted Gaps	9.39	33	7.89	46
Accepted Lags	10.74	116	8.52	149
Rejected Gaps	3.07	74	3.05	94
Rejected Lags	2.08	117	2.21	134

The logistic regressions for the southbound approach of the Fayetteville roundabout are shown in Figure 58 and in Figure 59. The parameters for the logistic equations along with the critical headway found from the equations are presented in

Table 18. The critical headway found when exiting vehicles are excluded in the analysis is 4.54 seconds. When exiting vehicles are included the critical headway is found to be 4.18 seconds. The difference between these two critical headway values is 0.36 seconds. Therefore, exiting vehicles are observed to affect the analysis at the Fayetteville roundabout but exiting vehicles do not have as great an effect at this roundabout as they do at the Covington roundabout.

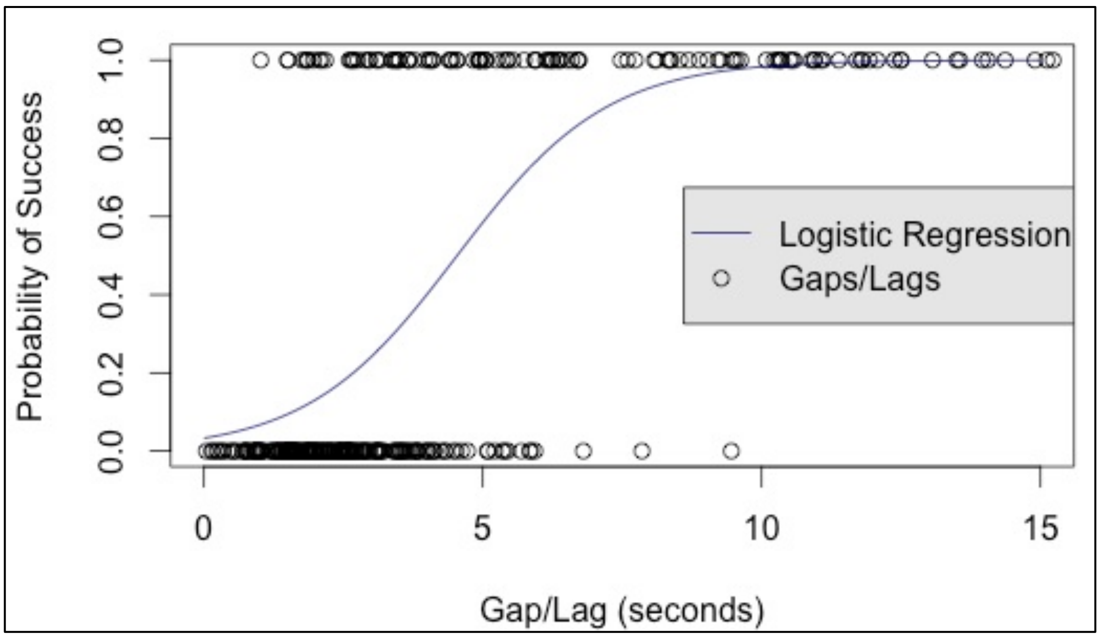

Figure 58. Fayetteville SB logistic regression excluding exiting vehicles

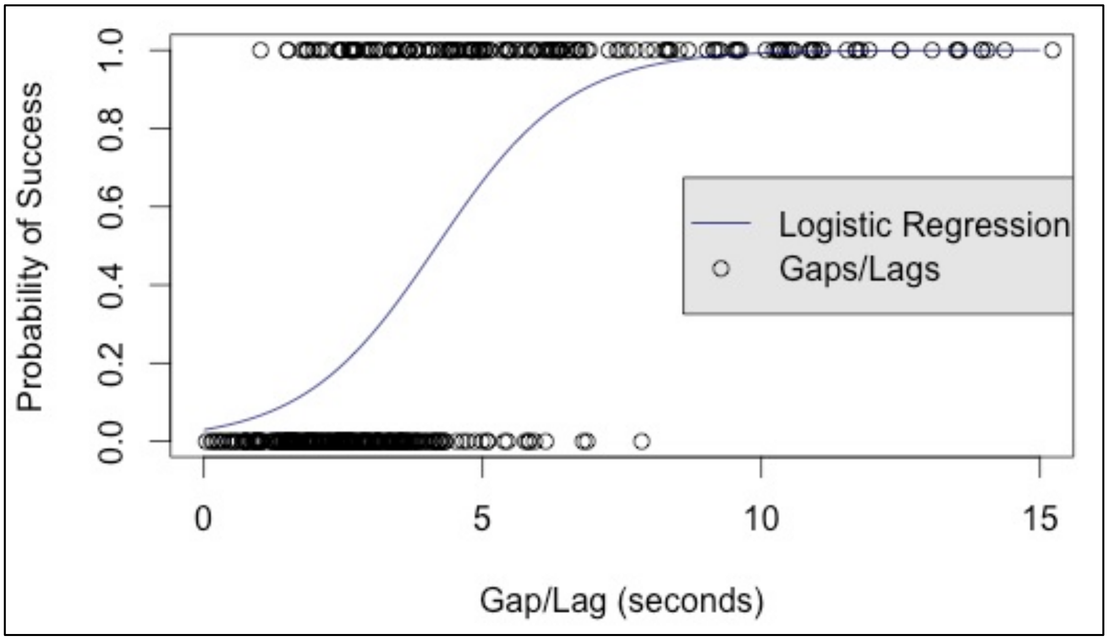

Figure 59. Fayetteville SB logistic regression including exiting vehicles

Table 18. Fayetteville SB logistic regression parameters and critical headway

Coefficients	Estimate	Std. Error	z value	Pr(>|z|)	Critical Headway (sec)
SB excluding exiting					
Intercept	-3.37978	0.36551	-9.247	<10e-6***	4.54
x	0.7446	0.09158	8.13	<10e-6***	
SB including exiting					
Intercept	-3.4911	0.3489	-10.005	<10e-6***	4.18
x	0.8354	0.0899	9.293	<10e-6***	

4.4 Comparison

Table 19 shows a comparison for follow-up headway and critical headway for the roundabout approaches considered in this study. Figure 60 and Figure 61 show graphical comparisons between the follow-up headway and the critical headway respectively, on the studied approaches.

Table 19. Comparison between Georgia Roundabouts

Location	Approach	Follow up Headway		Critical Headway	
		Excluding Exiting	Including Exiting	Excluding Exiting	Including Exiting
Covington	SB	3.40	2.83	4.73	2.90
	NB	3.75	2.80	4.91	3.28
Roswell	SB	3.03	2.74	3.75	3.82
	EB	3.30	2.60	3.47	3.38
Fayetteville	SB	3.12	2.98	4.54	4.18
NCHRP 572 Default		3.2		5.0	

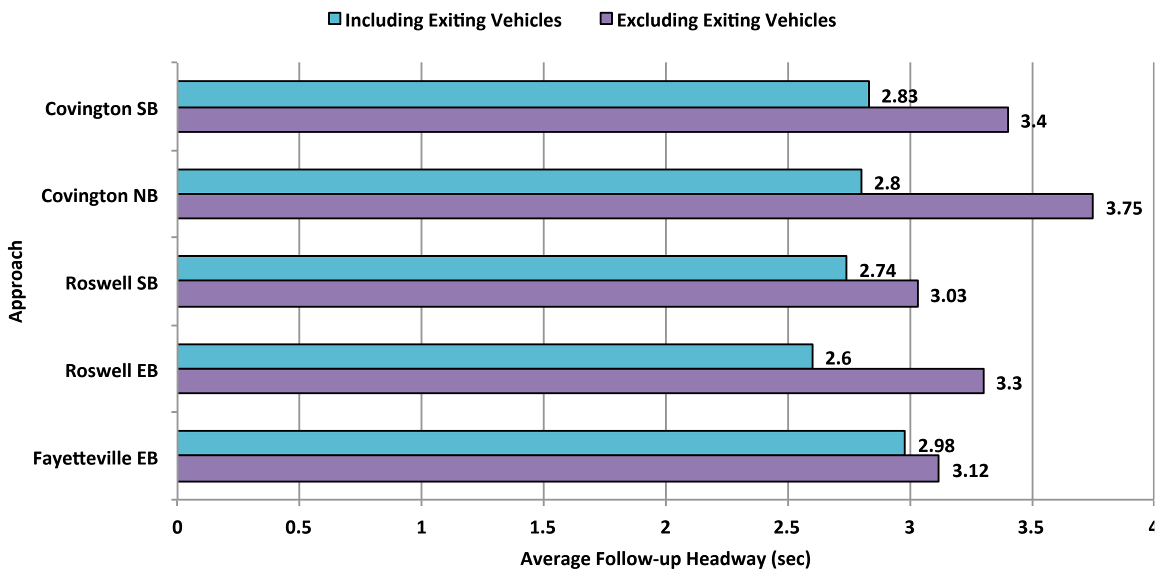

Figure 60. Comparison of Follow-up Headway

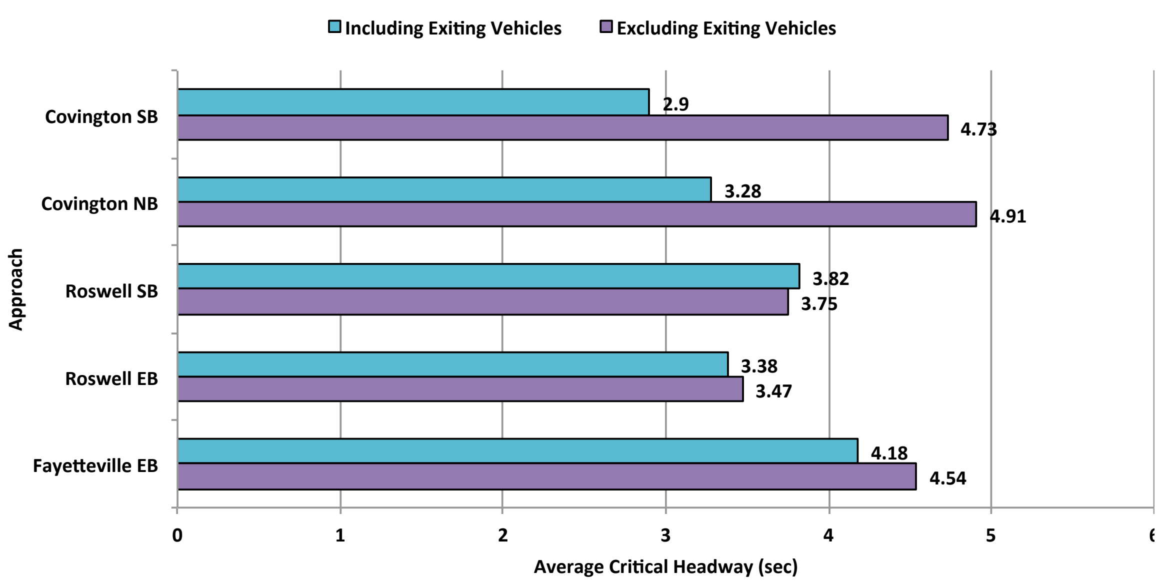

Figure 61. Comparison Critical Headway

When exiting vehicles are excluded, the approaches at the Covington roundabout have longer follow-up headways observed than the Fayetteville or Roswell roundabouts. However, when exiting vehicles are included in the analysis, the Fayetteville southbound approach has the longest follow-up headway, while the follow-up headway from both approaches at Covington and Roswell are very similar. When exiting vehicles are included, the follow-up headway observed at Covington is 2.8 seconds on both approaches. This is a decrease of 0.6 seconds on the southbound approach and 0.95 seconds on the northbound approach. Also, the eastbound approach of the Roswell roundabout sees a 0.7 second decrease in follow-up headway when exiting vehicles are included. The southbound approach of the Roswell roundabout and the southbound approach of the Fayetteville roundabout experience a small drop in follow-up headway when exiting vehicles are included, but it is less than is seen on the other approaches.

The change in follow-up headway seen when exiting vehicles are included in the analysis, is influenced by how many vehicles are exiting on the approach of interest. Table 20 shows the circulating vehicles and exiting vehicles by approach both by total number and by percent. Circulating vehicles are the vehicles that pass the approach of interest on the circulating roadway. Exiting vehicles are vehicles that exit the circular roadway on the approach of interest. The fraction of circulating vehicles was calculated by taking the number of circulating vehicles and dividing by the total number of conflicting vehicles, which is the sum of the circulating and the exiting vehicles. The percent exiting vehicles was found in the same manner. Three approaches, Covington SB, Covington NB, and Roswell EB all have less than 25% circulating vehicles, while the other two approaches have over 70% circulating traffic. From Figure 60, the approaches that see the greatest change in follow up headway are the same approaches that have less than 25% of the conflicting flow as circulating vehicles. The approaches with only a small percentage of the conflicting flow as exiting vehicles are not impacted as much by the inclusion of exiting vehicles in the analysis. Therefore, it is seen that exiting vehicles do have an impact on the follow-up headway on an approach, and the impact is determined in part by the proportion of exiting vehicles to circulating vehicles.

Table 20. Percentages of Exiting and circulating vehicles by approach

	# Circulating	% Circulating	# Exiting	% Exiting
Covington SB	438	22.5	1512	77.5
Covington NB	389	16.1	2023	83.9
Roswell SB	1114	72.9	415	27.1
Roswell EB	373	17.7	1732	82.3
Fayetteville SB	572	74.3	198	25.7

The Covington roundabout is dominated by the through movement on the north south oriented street and therefore, it was expected that the exiting vehicles would have an impact on the analysis. Neither the Roswell nor the Fayetteville roundabouts are dominated to the same extent by one movement. Therefore, it is expected that the Covington roundabout would see the greatest impact on follow-up headway by including exiting vehicles in the analysis. Additionally, Roswell is a much higher volume location than the Fayetteville roundabout thus is follows that the Roswell roundabout experienced a greater impact on follow-up headway from the inclusion of exiting vehicles than the Fayetteville roundabout.

The critical headway is also affected by exiting vehicles at most roundabout approaches. When exiting vehicles are excluded in the analysis, the smallest average critical headway is found at the Roswell roundabout, while the largest average critical headways are found at the Covington roundabout. However, when exiting vehicles are included in the analysis, the Covington roundabout has the shortest average critical headway. Since, the Covington roundabout is dominated by a through movement, it experiences the greatest effects from including exiting vehicles in the analysis.

The proportion of exiting vehicles to circulating vehicles appears to not affect critical headway as dramatically as it does follow-up headway. Both approaches at Covington, which also have high percentages of exiting vehicles, had a significant drop in critical headway due to the inclusion of exiting vehicles in the analysis. However, Roswell EB approach, which has 82.3% exiting vehicles, was found to have similar critical headway irrespective of whether or not exiting vehicles were included in the analysis. Therefore, the drop in critical headway at the Covington roundabout may be due to the drivers in the Covington roundabout driving more aggressively than drivers at the other roundabouts. Although, since the Fayetteville SB approach and the Roswell SB approach both have modest drops in critical headway due to exiting vehicles, the inclusion of exiting vehicles in the analysis may explain some of the drop in critical headway at the Covington roundabout.

There are several reasons that the drivers at Covington may be more aggressive than drivers at the other two roundabouts. First, the Covington roundabout was completed in 2010. By contrast both of the other roundabouts opened in 2011. Also, Covington has a larger inscribed diameter than the other two roundabouts and it is in a higher speed location. Lastly, there may be geometric differences between the roundabouts, such as the extreme angles between the roundabout legs at the Roswell roundabout, that make drivers more comfortable accepting smaller gaps at the Covington roundabout as compared to the other two.

Table 21 shows the average follow-up and critical headways found in the NCHRP 572 and Wisconsin reports. There are two numbers for the Wisconsin report because two roundabout approaches were studies and no overall average given for single-lane roundabouts. All of the follow-up headways found for the roundabout approaches included in this study fall between the upper and lower follow-up headways for the Wisconsin roundabout whether exiting vehicles are included or not. The critical headway found for the roundabouts in Georgia is generally lower than what is found for the Wisconsin roundabouts. These differences highlight the fact that calibration of the HCM 2010 equations is beneficial so that the results reflect the characteristics of local drivers.

Table 21. Averages from Wisconsin roundabouts and national data [22] [13]

		Follow-up Headway (seconds)	Critical Headway (Seconds)
NCHRP		3.2	5.0
Wisconsin	Ignoring Exiting	2.6, 3.8	5.5, 4.8
	Considering Exiting	2.4, 3.1	4.6, 3.8

The follow-up headway found for Georgia roundabouts is lower than the follow-up headway from the NCHRP 572 report, when exiting vehicles are included. When exiting vehicles are not included, the follow-up headway at three of the five approaches considered is greater than the average for the NCHRP 572 report. The calculation of the follow-up headway for the NCHRP report does not include exiting vehicles. The critical gap at Georgia roundabouts was found to be lower than was found in the NCHRP 572 report. Since, NCHRP 572 is based on data that was collected in the early 2000s the lower critical headway could indicate increasing familiarity of drivers with roundabouts. However, since the NCHRP 572 data does not include any data from Georgia, it is also possible that the national average does not accurately reflect Georgia drivers.

4.5 Equation Calibration

The HCM 2010 roundabout capacity equations require a value for follow-up headway and critical headway to be calibrated. However, two different follow-up headway values and two different critical headway values were found for each approach. Therefore, for comparison, the HCM 2010 roundabout capacity equation was calibrated for each set of values found. Figure 62. Graph of calibrated HCM 2010 equations is a graph of the HCM 2010 calibrated equations. The solid lines represent the analyses that include exiting vehicles and calibrated equations that exclude exiting vehicles are shown by dotted lines. Additionally, The HCM 2010 roundabout capacity equation for single-lane roundabouts is shown in black. Almost all of the calibrated equations have higher capacity than the HCM 2010 equation. Only the Covington northbound equation excluding exiting vehicles and the Covington southbound equation excluding exiting vehicles have lower capacity measurements and then only at low volumes of circulating flow. Generally, the equations that include exiting vehicles yield higher estimates of capacity than the equations that do not include exiting vehicles.

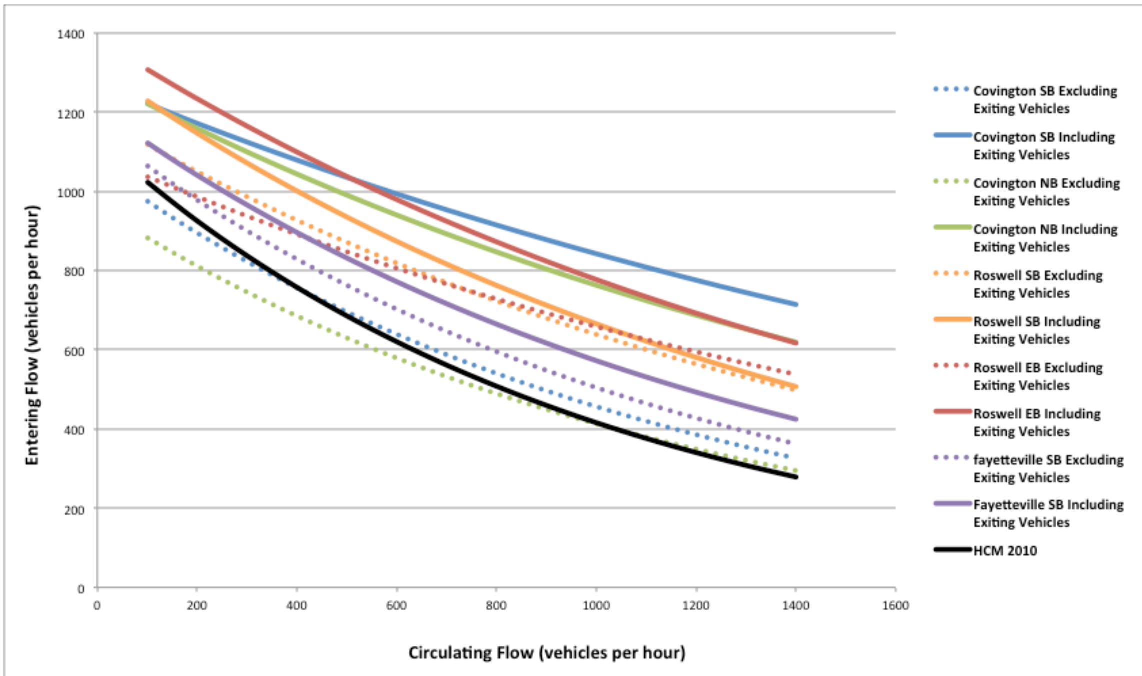

Figure 62. Graph of calibrated HCM 2010 equations

In order to calibrate the equations for Georgia, the weighted average of the critical headway and the follow-up headway was found for both the case where exiting vehicles were excluded and where they were included. The average was weighted using the number of data points for follow-up headway and critical headway found at each roundabout approach.

Table 22. Average headways for Georgia roundabouts

Average Follow-up Headway		Average Critical Headway	
Excluding Exiting (sec)	Including Exiting (sec)	Excluding Exiting (sec)	Including Exiting (sec)
3.46	2.81	4.22	3.34

Figure 63 shows the calibrated equations using average follow-up and critical headways. It is found that consideration of exiting vehicles in the analysis makes a difference. When exiting vehicles are included in the analysis, the capacity of the approach increases by approximately 200 vehicles per hour.

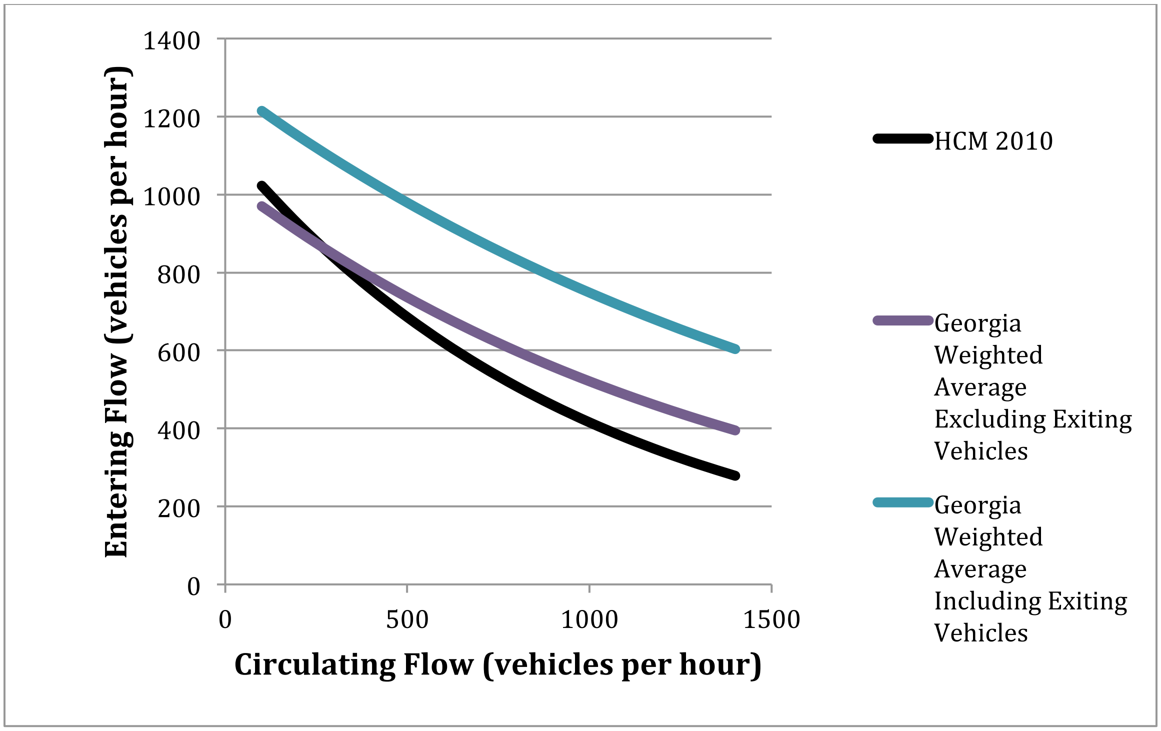

Figure 63. Calibrated equations using average follow-up and critical headways

Both of the curves in Figure 63 predict greater capacity than the HCM 2010 equation except for the curve that does not include exiting vehicles at very low conflicting flow rates. The equations for these two curves are shown as Equation 13 and Equation 14.

$$c = 1040 \cdot e^{0.000692 \cdot v} \qquad (13)$$

$$c = 1281 \cdot e^{0.000538 \cdot v} \qquad (14)$$

The difference between the HCM 2010 equation and the calibrated equations that include exiting vehicles is approximately 200 vehicles per hour for lower conflicting flow rates and around 300 vehicles per hour when the amount of conflicting flow increases. Figure 64 and Figure 65 show comparisons between the Georgia capacity equations found from this study and the equations found in other studies. Figure 64 shows the comparison between Georgia, California, and the City of Bend, Oregon. The California single-lane roundabout capacity equation is slightly steeper than the Georgia and the City of Bend Curves. Therefore, at very low conflicting flow rates, the California equation has the highest capacity. However, at conflicting flow rates over approximately 250 vehicles per hour, the Georgia equation including exiting vehicles predicts more capacity than the other equations. The City of Bend equation predicts capacity that is slightly higher than the weighted average for Georgia excluding exiting vehicles.

Figure 65 shows the comparison between the Georgia equations and the values found from the Wisconsin study. There are no calibrated equations given in the report from Wisconsin, but the curves were created using the follow-up and critical headway from each roundabout approach in Wisconsin. At conflicting flow rates of over 500 vehicles per hour, the weighted average equation for Georgia predicts higher capacity than all of the Wisconsin equations. Also, it is seen in this figure that all of the equations that include exiting vehicles predict higher capacity than their corresponding equation that excludes exiting vehicles.

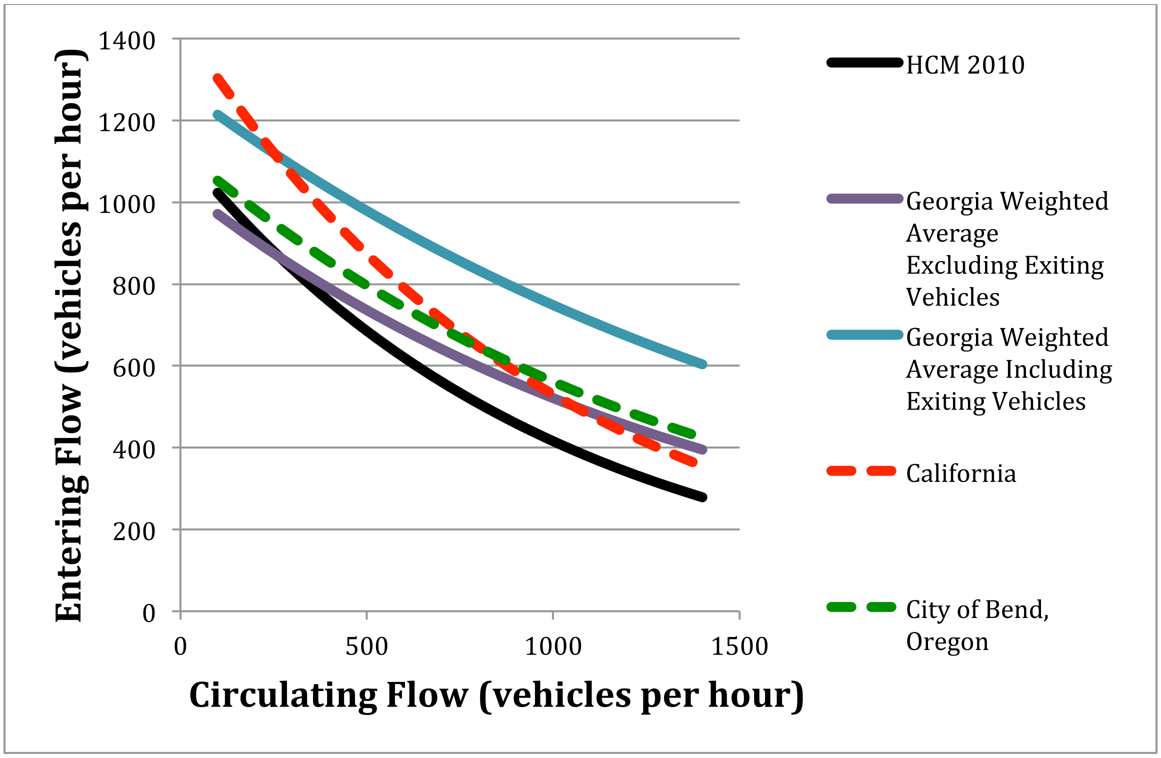

Figure 64. Comparison between Georgia, California, and the City of Bend, Oregon

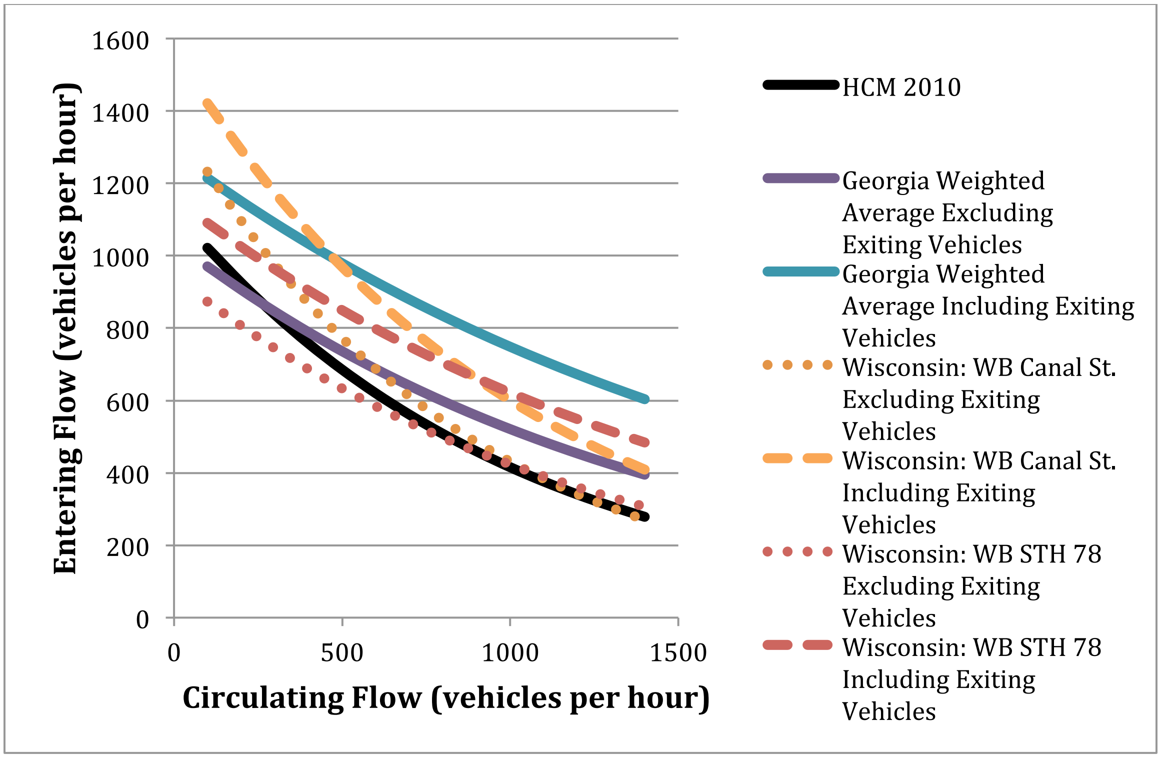

Figure 65. Comparison between Georgia and Wisconsin

There are several reasons that may explain why the HCM 2010 equations predict higher capacity when they are calibrated to Georgia conditions. There are three different methods for finding critical headway that are discussed in NCHRP 572, which were discussed in Chapter 2. The authors recommend using method 2, but there was found to not be enough data on some roundabout approaches to use method 2 or method 3 in this study. Therefore, method 1, which uses lags and gaps, was used. However, when the authors determined critical headway using method 1, it was found that the average critical headway is 4.5 seconds, while the critical headway was found to be 5.0 seconds using the preferred method 2. Therefore, it is possible that if sufficient data is obtained from roundabout approaches to allow for the use method 2, then the critical headway may have been higher, which would translate into an equation that predicts lower capacity. Future efforts will seek to obtain additional data to implement a method 2 calibration.

Another factor that could be contributing to the increased capacity predicted by the calibrated equations is that the lags were not adjusted. According to NCHRP 572, "The lags have been converted to gaps using an approximate follow-up headway" [13]. There is no follow-up headway added to the lags used for this study, because there is no more information given on how the "approximate follow-up headway" was determined or how it was applied to the lags. However, adding time to lags could increase the critical headway calculated at the roundabout approaches, which would in turn lead to and equation that predicts less capacity than Equation 13 and Equation 14.

Exiting vehicles are another factor that affects the capacity of the approach. In this study, some vehicles were observed to hesitate when entering the roundabout when a vehicle was exiting the roundabout on the same leg. Including these vehicles creates shorter gaps because the gaps created by the exiting vehicles may break up several longer gaps that existed when only circulating vehicles were included in the analysis. As can be seen from Figure 63, when exiting vehicles are included there is a noticeable change in the capacity predicted between the HCM 2010 equation and the calibrated equation. The difference between the calibrated excluding exiting vehicles and the HCM 2010 equation is less significant. As exiting vehicles were not considered in the calibrated equations found in the HCM 2010 these findings appears reasonable.

CHAPTER 5

CONCLUSION, LIMITATIONS, & FURTHER STUDY

5.1 Conclusion

The purpose of this study was to calibrate the HCM 2010 roundabout capacity equations to Georgia conditions, focusing on the single lane roundabout equation. A list of roundabouts in Georgia was prepared. Data were collected at six roundabouts, chosen for favorable AADT, location, geometry, and other characteristics. However, insufficient traffic was observed at three of the roundabouts and therefore only data from the Covington, Fayetteville, and Roswell were used for subsequent analysis. Videos cameras were used to record operations at the roundabouts in the field for approximately two hours during the AM/PM peak periods. The resulting videos were then analyzed in the lab using several programs to extract the follow-up headway and the critical headway. The follow-up headway and critical headway were subsequently used to calibrate the HCM 2010 roundabout capacity equations for Georgia.

It was also found that vehicles exiting the roundabout had a measurable impact on the capacity of roundabouts in Georgia. However, the impact varied depending on the volume of exiting vehicles. Roundabouts with high percentages of the conflicting flow as exiting vehicles saw a larger change in follow-up headway than roundabouts whose conflicting flow was made up primarily of circulating vehicles. Critical headway also tended to decrease when exiting vehicles were included in the analysis. The inclusion of exiting vehicles in the analysis of the Covington roundabout caused a substantial drop in the critical headway. Since the change at Covington was much greater than seen at any of the other roundabout approaches, the drop may also be partially attributed to variations in driver aggressiveness. Lastly it was found that calibrating the HCM 2010 roundabout capacity equations to Georgia conditions generally increased the predicted capacity. When exiting vehicles were ignored, the capacity only increased by about 100 vehicles per hour at high conflicting flows and was slightly lower than the HCM 2010 model at conflicting flows of 300 vehicles per hour or less. When exiting vehicles were considered in the analysis the predicted capacity increased by between 200 and 300 vehicles per hour depending on the conflicting flow rate when exiting vehicles were considered in the analysis.

5.2 Limitations

There are several limitations to this study. One limitation of this study is that there are few roundabouts in Georgia particularly that have been in operation for more than a year. Even though the number of roundabouts is increasing, there are still very few roundabouts that are in locations operating near capacity traffic conditions, which is need for capacity equation calibration. If the approaches only experience queuing for a short amount of time, then there is limited follow-up headway data that can be measured at that roundabout. Similarly, if the volume of conflicting traffic is low then there are not many observations for the critical headway calculation.

Another limitation of this study is that the roundabouts included are extremely new. Both the Roswell and Fayetteville roundabouts have be in for a year or less. Only the Covington roundabout has been in place for more than a year and it was constructed less than two years ago.

A third limitation of this study is the lack of gap data available at current roundabouts. NCHRP 527 has three methods for determining follow-up headway and the authors recommend the second method. However, this method does not use lags and there is insufficient data on a number of the roundabout approaches in Georgia to allow for omitting lags in the analysis. Therefore, method 1 must be used.

5.3 Further Study

There are several options that could be explored for further study. First, the effect of trucks at roundabouts in Georgia could be studied. The Fayetteville roundabout along with a few of the other roundabouts where data was collected had very low truck volumes. However, the Covington roundabout was observed to have some large trucks. The analysis could be refined to account for the effect of trucks and other heavy vehicles if this distinction were made. None of the roundabouts considered in this study had high volumes of pedestrians or bicycles but studying the effect of these users at roundabouts in Georgia could also be beneficial to capacity estimation.

Another option that could be explored for further study is the calibration of the HCM 2010 multilane roundabout capacity equations. This study focused on the single lane capacity equations because single lane roundabouts are much more common in Georgia than multilane roundabouts. The only multilane roundabout in Georgia is located in St. Simons Island [17]. Therefore, it would be beneficial for a multilane study to occur after more multilane roundabouts are constructed in Georgia. Calibrating the HCM 2010 multilane roundabout capacity equations would help with informed decision making regarding a multilane roundabout in the state.

Further research should focus on collecting data at more roundabouts. This study only used data from five approaches at three different roundabouts. However, it would be beneficial to refine the calibrated equations by using data from more roundabouts in

Georgia. Additionally roundabouts in neighboring states should also be considered for future data collection efforts.

Another area of future research is reconsidering how exiting vehicles are included in the analysis. As was seen at the Roswell roundabout, the critical headway increased slightly when exiting vehicles were included. This finding most likely occurred because exiting vehicles do no have a huge impact at this roundabout approach and because of the way that the exiting vehicles were projected to the conflicting point when measuring the perceived gap. Only the second vehicle in each gap was projected if it was an exiting vehicle. However, not projecting the first vehicle in the gap if it was an exiting vehicle makes these gaps longer than they should be. Going forward, the way that exiting vehicles are included in the analysis should be revisited and reevaluated. Also the effects of geometry on a driver's perception of exiting vehicles should also be considered in the reevaluation of including exiting vehicles in the analysis because different geometric conditions could cause drivers to be either more or less likely to hesitate because of an exiting driver.

Lastly, and area of further study that would be beneficial is repeating the calibrations of the single lane roundabout capacity equations sometime in the future. In several years there will be many more roundabouts in the state than there are now. Also current roundabouts and the roundabouts included in this study will have been in place longer. Therefore, repeating the calibration in several years will allow for the determination of whether or not roundabout capacity changes over time. If the roundabout capacity has changed as a result of drivers becoming more used to the intersection type, the repeating the calibration will allow for capacity estimation to stay current and allow decision makers to make more informed decisions as more and more roundabouts are built in the state.

APPENDIX A

TIMESTAMP LOCATIONS

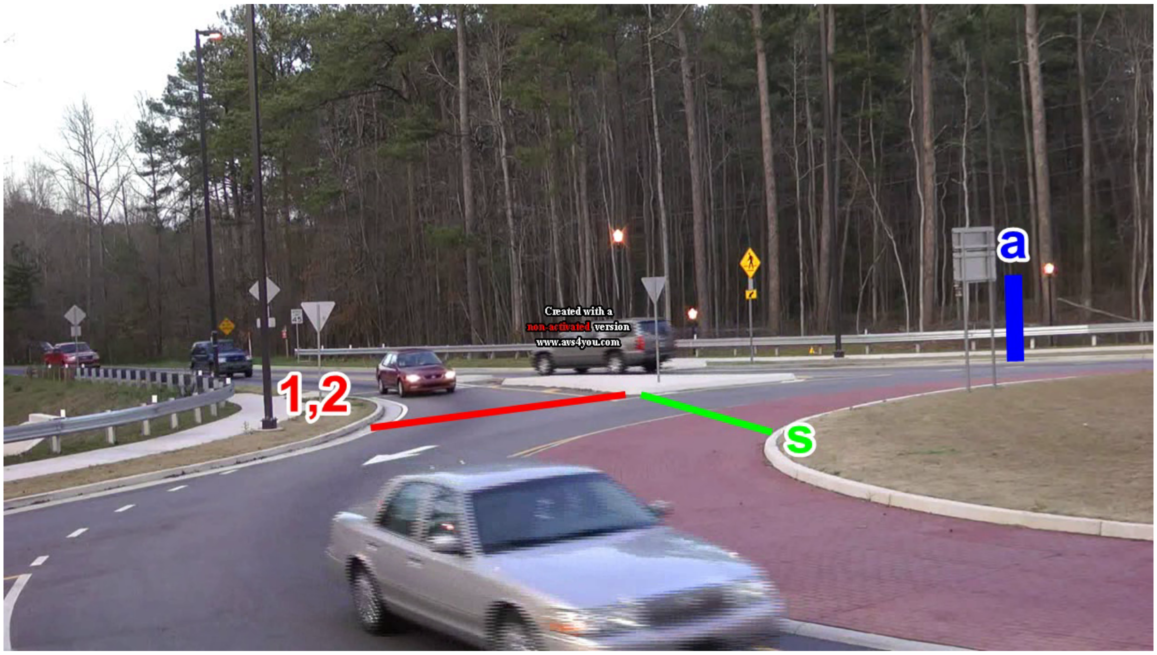

Figure 66. Timestamp locations for Covington SB approach

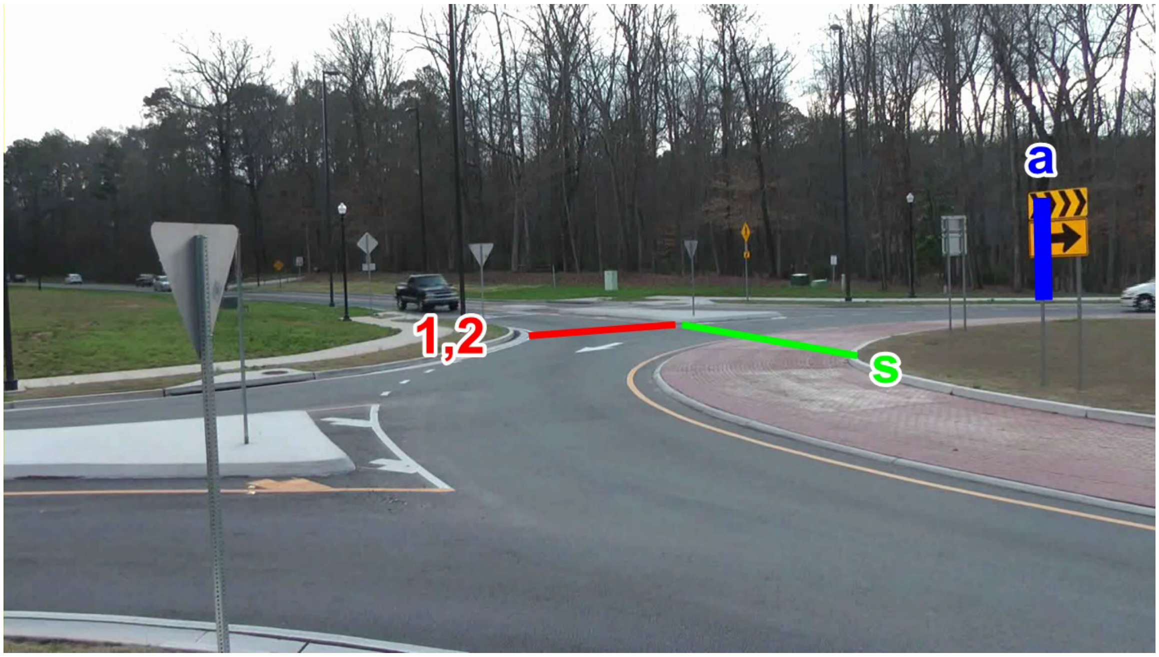

Figure 67. Timestamp locations for Covington NB approach

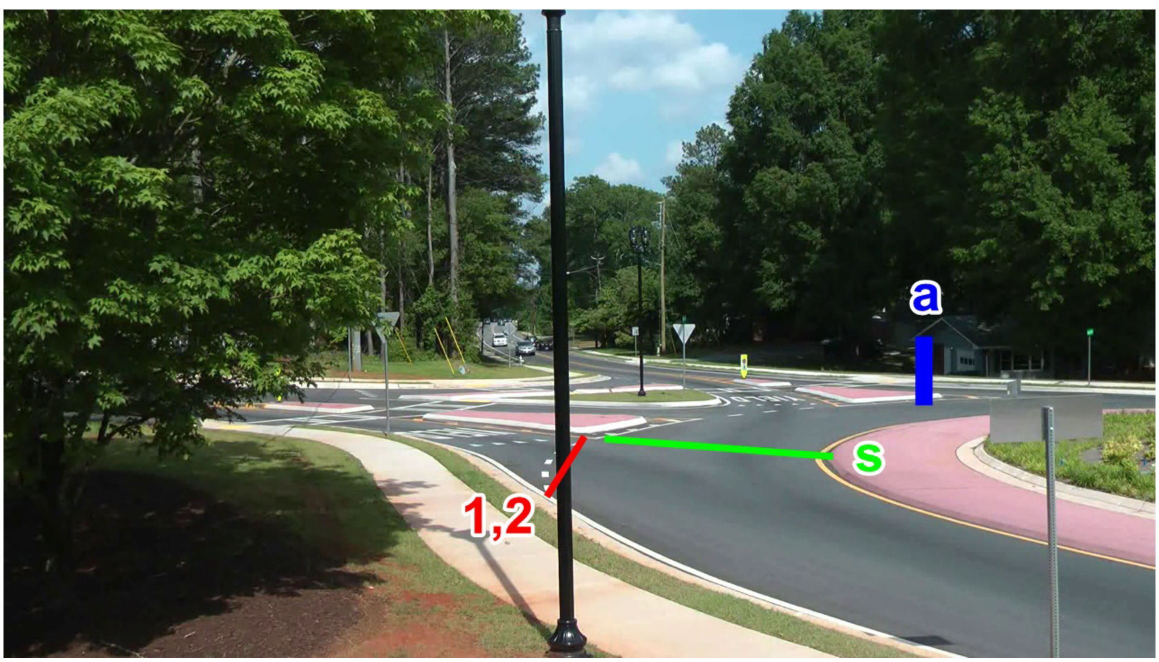

Figure 68. Timestamp locations for Roswell SB approach

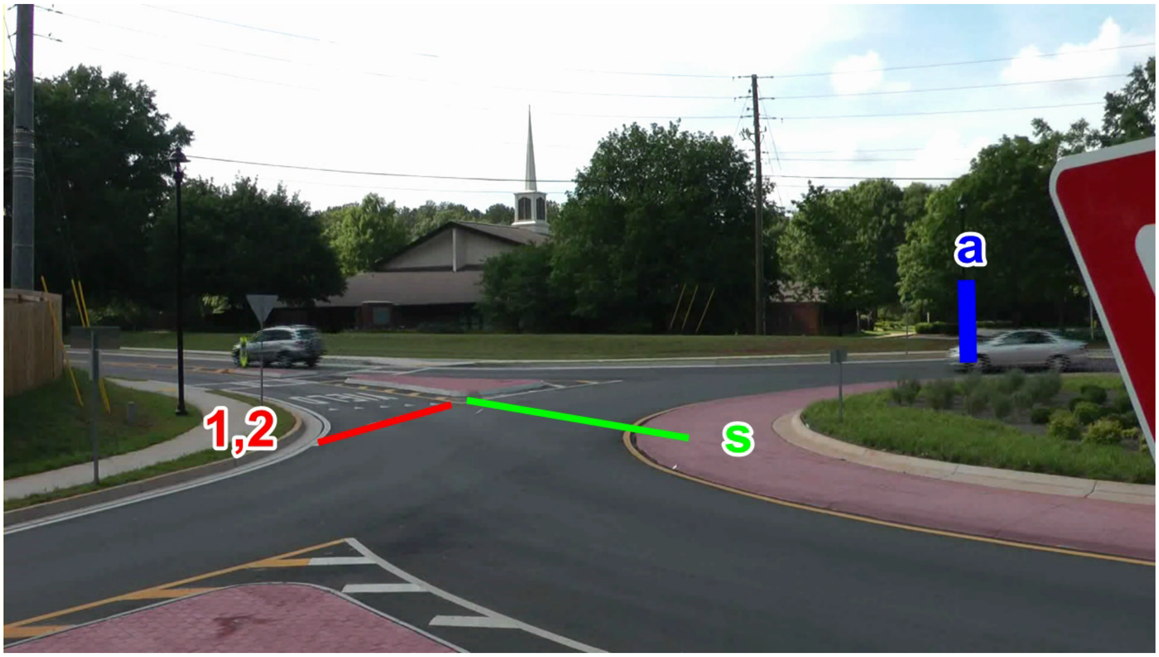

Figure 69. Timestamp locations for Roswell EB approach

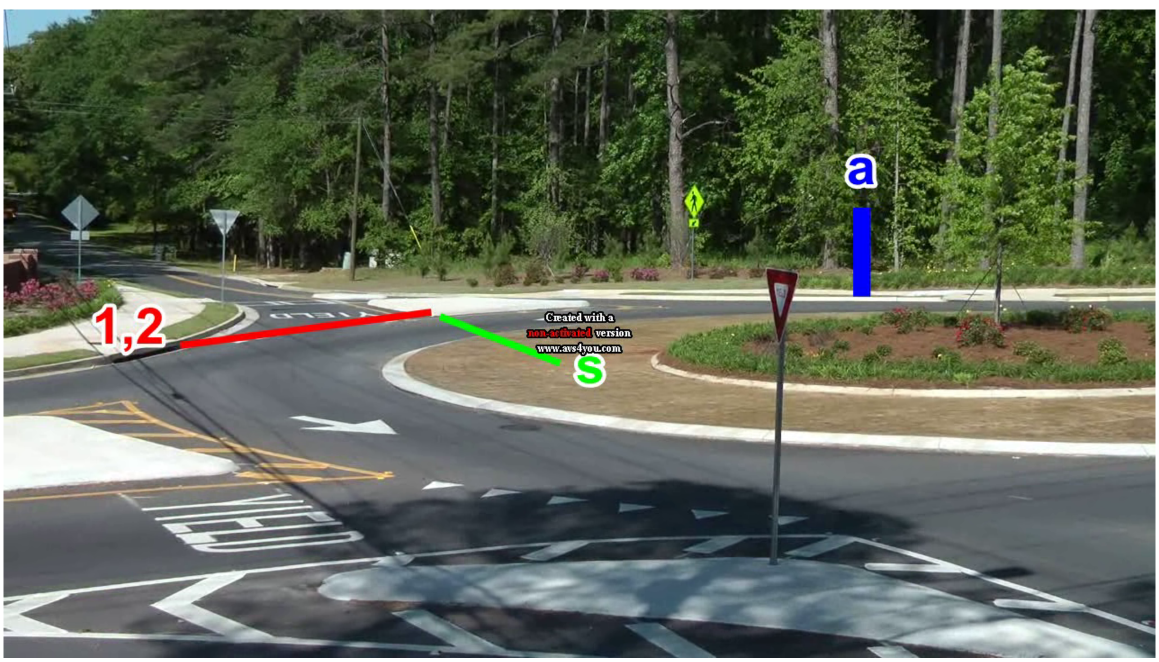

Figure 70. Timestamp locations for Fayetteville SB approach

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